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Joints in Steel Construction
Composite Connections

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FOREWORD

This publication is one in a series of books that cover a range of structural steelwork connections. It provides a guide to the design of Composite Connections in Steelwork. Other books in the series are *Joints in simple construction*, Volumes 1 and 2 (shortly to be replaced by *Joints in steel construction - Simple Connections*), and *Joints in steel construction - Moment Connections*.

This guide includes composite end plate connections suitable for use in semi-continuous braced frames. Both beam-to-column and beam-to-beam details are considered. Guidance on frame design procedures is also given.

The publication begins with a list of 'Fundamentals'. These points should be clearly understood by anyone wishing to design a frame incorporating composite connections.

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

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FUNDAMENTALS

Designers should ensure that they have a good understanding of the following 'Fundamentals' before starting to design composite connections using this guide.

In order to keep design procedures simple, a number of issues (e.g. connection rotation capacity) are not checked explicitly. In some cases detailing limitations are given in preference to complicated checks in order to ensure that connection behaviour is appropriate. A good understanding of these 'Fundamentals' will help a practising engineer to appreciate some of the background to these requirements, without a need to employ overly complicated checks.

Mechanics of composite connections

Composite connections resist moment by generating a couple between their tension and compression components. The mechanics are essentially the same as those for bare steel moment connections, with the slab reinforcement acting like an additional row of bolts in an extended end plate. In order to achieve their full potential, the reinforcing bars must be properly anchored, and be capable of accommodating significant strain before fracture.

It may be assumed that the lower beam flange can sustain a stress of $1.4p_y$ in compression when it is assumed to act alone. When part of the beam web is also assumed to be subject to compression, the limiting stress should be reduced to $1.2p_y$. Compression often extends into the beam web in composite connections as a result of high tensile forces in the reinforcement. A further consequence of these high forces is that column compression stiffeners are often required.

Detailing

Considerable care is needed when detailing composite connections to ensure that components are subjected to sufficient deformation to allow them to generate their full potential resistance, whilst at the same time ensuring that they are not over-strained to the point of premature failure.

Detailing rules given in this guide ensure that the full potential resistance of bolt rows that are too near the neutral axis is not considered in the calculation of moment capacity. Similarly, to ensure that sufficient strain takes place to yield the reinforcement, compression must be limited to the lower half of the steel beam. To prevent premature failure of the reinforcement (due to excessive strain) adversely affecting the connection's rotation capacity, it is also essential that reinforcing bars are not located too far from the neutral axis.

Detailing rules are given for two basic types of connection. Less onerous rules, in terms of the minimum area of reinforcement required, lead to what may be described as 'compact' connections. Like 'compact' beams, these connections can develop a moment capacity that is based on a stress block model (analogous to M_p for a beam), but have insufficient rotation capacity to form a plastic hinge. More onerous limitations are needed for 'plastic' connections, which are capable of forming a plastic hinge.

Non-ductile failure modes must not govern the moment capacity of 'plastic' connections. These include:

- column and beam web tension failure
- column web buckling or bearing failure in compression.

Non-ductile failure modes must be avoided either by local stiffening/strengthening, or by modification of component choice.

All composite connections detailed in accordance with this guide will be 'partial strength', i.e. their moment capacity will be less than that in hogging of the beam to which they are attached.

All connections detailed in accordance with this guide will be 'rigid' in their composite state.

Materials

The properties of the reinforcement used in a composite connection, in particular the elongation that the reinforcement can undergo before failure, are of vital importance because they have an overriding influence on the rotation capacity of the connection. Designers should note the following points in relation to reinforcement ductility:

- The contribution of any mesh to the moment capacity of the connection should be ignored, as mesh may fracture before the connection has undergone sufficient rotation at the ultimate limit state. Structural reinforcement should comprise 16 mm or 20 mm diameter bars.

- Reinforcing bars currently produced in the UK are often considerably more ductile than those specified in either BS 4449 or BS EN 10080. Detailing rules are therefore given for two cases; bars that just meet the code requirements (identified as bars that are capable of 5% total elongation at maximum force - see Section 4.2 Step 1A for exact definitions of code requirements), and bars that have twice this elongation capacity (10%). When the designer has assumed that bars can achieve 10% elongation, he must make this non-standard condition clear in the contract documents. Bars with non-standard performance requirements should be identified with an X (i.e. X16 or X20) to indicate that specific requirements are given in the contract documents.

Steelwork detailing must also ensure that adequate rotation can take place. To achieve this, rotation should be primarily the result of end plate or column flange bending, rather than by elongation of the bolts or deformation of the welds, as these components generally fail in a brittle manner. End plates should always be grade S275, regardless of the beam grade.

Frame design

Recommended frame design procedures, considering both the ultimate (ULS) and serviceability (SLS) limit states, are given in this publication. Beam design at the ULS assumes that plastic hinges form in the connections at the beam ends. The method is therefore only applicable when 'plastic' connections are used. In addition, the following limitations must be imposed to ensure that the required beam end rotations are not excessive:

- a minimum required connection strength (30% relative to the beam in sagging),
- a lower bound on the beam span to depth ratio,
- a reduction factor on the sagging moment capacity of the composite beam. Although in theory this reduction factor varies as a function of several parameters, including the beam grade and load arrangement, a value of 0.85 may be used for all cases. The reduction factor is necessary to limit the amount of plastification that takes place in the beam, and thereby substantially reduce the end rotation requirements.

Implications of propping the beams during construction are far reaching, and considered at some length. Not only will dead load deflections clearly be affected, but there will also be an influence on the moments that are applied to the columns, and the levels of rotation required from the connections. The implications of propping can even affect the basic choice of frame layout and connection types. The designer must therefore clearly communicate his requirements for propping to all parties concerned.

1 INTRODUCTION

1.1 ABOUT THIS DESIGN GUIDE

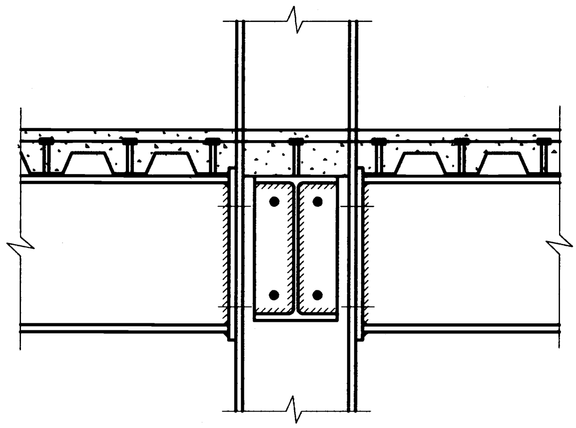
Composite construction has achieved dominance in the UK because of its overall economy of use of materials, and ease of construction relative to alternative reinforced concrete and steel options. Attention has now turned to further improving the economy of composite construction by taking advantage of the performance of the connections in the analysis and design of the frame. Even relatively simple non-composite details can achieve a reasonable degree of stiffness and strength when composite action is present. This is not only due to the continuity of reinforcement in the slab, but is also the result of other less quantifiable effects, such as membrane action in the floor plate.

This publication considers connections in frames where the steel beams act compositely with concrete floor slabs, and where some of the connections are also designed to act compositely. The structural interaction of the beams and slabs allows smaller beams to be used in a frame of given stiffness and strength. Shear connectors provide the means of enhancing moment capacity and stiffness by transferring longitudinal shear between the steel beams and concrete. In addition to the beams, the floor slabs themselves are often composite, comprising profiled steel decking and in-situ concrete. However, most of the beam-to-column and beam-to-beam connections in composite frames are currently non-composite and treated as 'simple'. Their ability to resist moment is not exploited, mainly because of a lack of appropriate design guidance for composite connections.

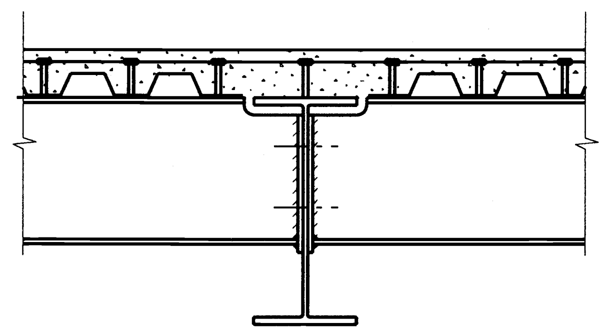
Procedures for the design of moment resisting composite connections, and guidance on the layout and design of braced semi-continuous frames incorporating composite connections, are given in this publication. Typical composite connection details are shown in Figure 1.1. Reinforcement comprises 16 or 20 mm bars local to the column. Composite interaction between the steel components and the reinforcement, via the concrete slab, enables the connection to resist moment by forming a couple between the reinforcement in tension and steel beam in compression.

The most cost effective composite construction will often arise when composite connections are used in conjunction with moment resisting non-composite connections in appropriate locations. For maximum economy, the designer should also consider the use of some 'simple' steel details, for example connections to perimeter columns in order to prevent the transfer of substantial moments.

Design procedures for bare steel connections are not included in this publication. The designer should refer to other books in the BCSA/SCI 'Green Book' series for information concerning bare steel details[1,2,3,4].



(a) beam-to-column



(b) beam-to-beam

Figure 1.1 Typical composite connection

1.2 SCOPE

This publication covers the following types of connections:

- composite, using flush end plates for beam-to-column connections
- composite, using partial depth end plates for beam-to-beam connections
- suitable for use in braced frames only
- rigid

- *either* 'compact' - so that their moment capacity can be calculated using plastic stress blocks
- *or* 'plastic' - in addition, they have sufficient rotation capacity to be justifiably modelled as plastic hinges.

A range of standard connections is included, all of which are 'plastic', and which are therefore suitable for inclusion in a frame that is analysed using plastic methods. Steelwork detailing for these connections is based on the standard wind-moment connections presented in Reference 4.

The standard composite connections have all been detailed so that the connection moment capacity is less than that of the beam (in hogging). This makes all the standard connections *partial strength*. This is a necessary requirement when plastic frame analysis is adopted and the beams are anything other than Class 1 (plastic) in hogging, because the plastic hinges form in the weaker connections rather than the adjacent beams.

Because plastic models are used to calculate the moment capacity, beam flanges must be either Class 1 or 2, and webs Class 1, 2 or 3. Class 3 webs may be treated as effective Class 2[5].

Although the publication is essentially a composite connection design guide, it includes guidance on choice of frame topology and frame design procedures.

1.3 BENEFITS

The use of composite connections in braced frames can result in:

- reduced beam depths, which may be important for the integration of building services, reduction in overall building height, reduction in cladding costs etc.
- reduced beam weights
- improved serviceability performance
- greater robustness, as a result of improved continuity between frame members (see Section 2.6)
- control of cracking in floor slabs on column lines (due to the presence of substantial reinforcement).

For a semi-continuous composite frame, that is one in which the connections are partial strength, the weight and depth savings on individual beams may be up to 25%. Overall frame savings in weight and depth will vary considerably depending on the extent to which an optimal framing arrangement can be adopted. Guidance on framing arrangements which exploit the benefits of composite connections is given in Section 1.4.

For maximum cost savings it is essential to base the composite connections on steel details that are not significantly more complicated than those traditionally considered to be 'simple'. Column tension stiffeners not only increase fabrication costs, but may also complicate the positioning of other incoming beams. They should therefore be avoided where possible. Although column compression stiffeners also increase fabrication costs and should preferably be avoided, they are less of a problem if the orthogonal beams are sufficiently shallow to avoid a clash. Compression stiffeners are often unavoidable in composite connections that adopt a substantial area of reinforcement. In some situations increasing the column size may be more cost effective than local stiffening.

1.4 FRAME LAYOUT

The extent to which a beneficial framing layout can be adopted will have a major influence on whether the use of composite connections is economical. General principles that should be considered when planning the layout of beams and orientation of columns are given in this Section.

1.4.1 Unpropped construction

Beam design criteria

Beam size may be governed by any of the following factors:

- the strength of the bare steel beam during construction
- the stiffness of the bare steel beam during construction
- the strength of the composite beam in its final state
- the stiffness of the composite beam in its final state.

The second of these is only relevant if dead load deflections need to be controlled (for example to prevent excess ponding of the concrete during casting) and pre-cambering of the steel beam is not a viable option.

Stiffness considerations are most likely to be critical for beams that are made from higher grade steel (S355) and that are subject to UDL or multiple point loads. Stiffness influences both deflections and response to dynamic loading.

Choice of connections

Beams that are governed by strength considerations need strong (moment resisting) connections to reduce the sagging moment that must be resisted by the beam. This applies to either the bare steel beam (with steel connections) during construction, or the composite beam (with composite connections) in its final state.

Beams that are governed by stiffness considerations need stiff connections to provide rotational restraint at the beam ends. This could be in either the initial bare steel or final composite state. In the absence of stiff bare steel connections, pre-cambering can be used to reduce dead load deflections of unpropped beams during construction.

The following guidelines should also be considered when choosing the connections:

- The use of both major and minor axis composite connections to a given column may result in problems accommodating the necessary reinforcement in a limited thickness of slab. It is generally recommended that the connections to one of the axes should be non-composite.
- It is recommended that connections to perimeter columns should be non-composite, to avoid problems anchoringor locating the reinforcement.
- Beam-to-beam connections based on partial depth end plate steel details offer limited stiffness at the construction stage. If dead load deflections of secondary beams are to be controlled, pre-cambering may be the only possible option (in the absence of propping).
- For 'two sided' composite connections, connecting the steelwork to the column web avoids the common need for local column stiffening.

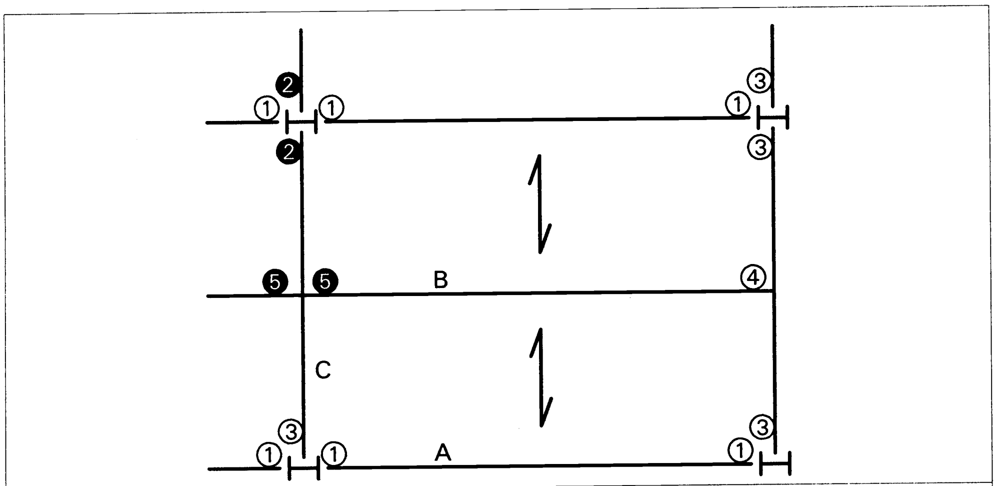
Typical framing solutions

Two framing solutions that capitalise on the principles outlined above, and avoid the need for propping, are shown in Figures 1.2 and 1.3. Schematic connection details, and references to where appropriate design guidance may be found, are shown on each Figure. These details are included to illustrate the required connection characteristics at various frame locations, for example non-composite and 'simple', or composite and moment resisting. The types of detail shown will not be appropriate in all situations, for example flexible end plates may need to be used for 'simple' connections, rather than fin plates, when beams have a limited web area.

The following points explain the choice of layout shown in Figure 1.2.

- The use of moment resisting composite connections (type 2) on the beams C (which are governed by strength considerations) allows the sagging moment requirements on the composite beams in the final state to be reduced.
- The use of moment resisting non-composite connections (type 1) on the beams A (which are governed by stiffness considerations) allows the dead load deflections to be reduced, without the need for pre-cambering. It also avoids a clash with the orthogonal composite connections at the internal columns.
- The type 3 non-composite connection details have been chosen at the perimeter ends of beams C to facilitate erection, and to avoid the complexities of producing moment resisting non-composite details to the webs of perimeter columns.
- The type 4 non-composite connection at the perimeter end of beam B is chosen for ease of erection, and because the torsionally weak perimeter beam offers no moment resistance capability.
- The type 5 composite connections can be used at one end of beams B to reduce sagging moments and imposed load deflections in the final state. Dead load deflection of these beams will normally only be reduced if pre-cambering is adopted, because of difficulties in achieving stiff bare steel beam-to-beam connections during construction. Type 5 composite connections will be difficult to achieve should the secondary beams (B) not be shallower than the primaries (C). This will depend on the relative spans. Pre-cambering would therefore become more attractive as the span of the mark B beams increased.

Figure 1.3 shows another potential framing solution, for which the long span beams will be deeper/heavier than would be the case for an arrangement as shown in Figure 1.2.



① Non-composite
Moment resisting
Reference 4, Section 2.8

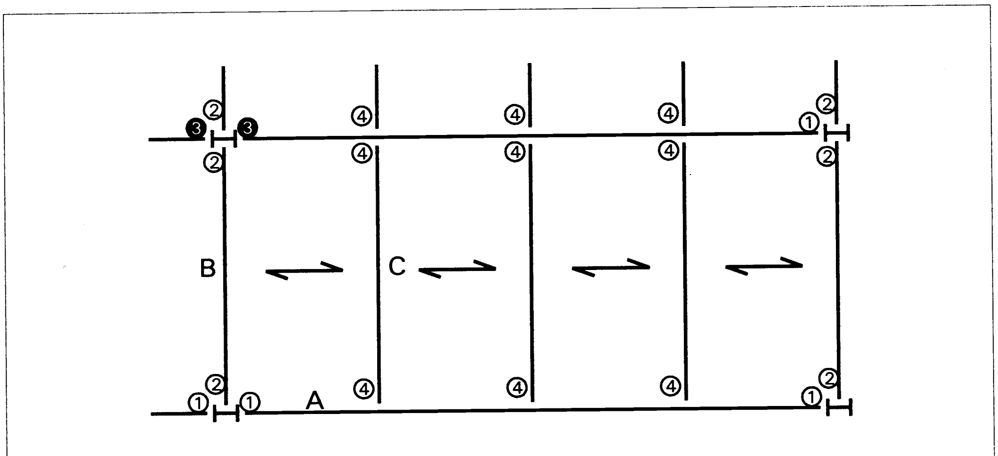
② **Composite**
Moment resisting
Section 4.2

③ Non-composite
Nominal pin
Reference 1, Section 3.5

④ Non-composite
Nominal pin
Reference 1, Section 4.5

⑤ **Composite**
Moment resisting
Section 4.3

Figure 1.2 Typical floor beam layout - for spans up to approximately 12 m it may be possible to achieve beams of similar depth. (Numbers represent connection types. A, B and C are beams)



① Non-composite
Moment resisting
Reference 4, Section 2.8

② Non-composite
Nominal pin
Reference 1, Section 3.5

③ **Composite**
Moment resisting
Section 4.2

④ Non-composite
Nominal pin
Reference 1, Section 4.5

Figure 1.3 Alternative floor beam layout - services may be located beneath short span beams. (Numbers represent connection types. A, B and C are beams).

- The use of moment resisting connections, either composite (type 3) or non-composite (type 1) allows the size of the long span beams A to be reduced.
- The depth of the short span beams B and C will be significantly less than that of the long span members, so there will be no penalty in terms of overall depth of the steel members if the designer adopts inexpensive 'simple' connections for these beams.
- It may be possible to run services beneath the secondary beams, within the depth of the long span primaries.

The economics of this type of solution will depend on the relative spans, but it will normally be less efficient than a layout of the type shown in Figure 1.2.

1.4.2 Propped construction

If propping is acceptable (recognising that there may be implications in terms of both the work programme and costs), or there is no need to control dead load deflections, the most economic frame layouts may differ from those shown in Figures 1.2 and 1.3. Composite connections should be used in the most beneficial locations, either to reduce sagging moments, or to reduce imposed load deflections. These improvements are possible because of the strength and stiffness of the connections respectively. The most appropriate connection choices will depend on the relative spans, and the column orientations will be influenced by the connection choice.

1.5 FRAME DESIGN METHODS

It is essential, and indeed a requirement of BS 5950: Part 1[(6)], that connection behaviour is compatible with the assumptions made in the design of a frame. The connection characteristics determine whether the frame is:

- Simple - the small moment capacity and stiffness that the connections possess are neglected in the frame analysis, and the connections are treated as 'pinned'.
- Semi-continuous - the moment capacity of the connections is allowed for in a plastic global analysis of the frame. Alternatively, their stiffness is allowed for in an elastic analysis. Both connection stiffness and strength are considered in an elastic-plastic analysis of a semi-continuous frame.
- Continuous - the connections are designed to resist moments and forces predicted by an elastic or plastic global analysis, assuming that they either behave rigidly (elastic analysis) or are 'full strength' (plastic analysis), to provide full continuity between the frame members. Rigid, full strength behaviour is assumed in an elastic-plastic analysis of a continuous frame.

Global analysis - ULS

In a semi-continuous frame the moments and forces at the ULS may be determined by either elastic or plastic global analysis. Factors that influence the choice of analysis method are:

- the type of connections used
- the classification of the beam cross-sections (noting differences between hogging and sagging).

Elastic frame analysis relies on the assumption that each material being modelled behaves in a linear-elastic manner. An appropriate value of elastic modulus must be used, so that member stiffness can be calculated. When connections are semi-rigid, their stiffness must also be incorporated in the analysis. Procedures are given in EC3 Annex J[(7)] for bare steel connections, and the COST CI document[(8)] that will form the basis for the EC4 Annex for composite connections, for calculating stiffness.

Rigid-plastic analysis considers the resistance of members and connections rather than their stiffness. This avoids the need to predict connection stiffness. Connection strength, i.e. moment resistance, can be predicted more accurately than stiffness using current methods. A rigid-plastic analysis assumes that plastic hinges form at certain points in the frame. This assumption is only valid when the points at which hinges may form, including the connections, have sufficient capacity to rotate without loss of strength.

A third possibility is to combine stiffness and strength considerations in an elastic-plastic analysis. Software may be used to perform this type of analysis, allowing for the connection characteristics. Although such software is not common in design offices, it is used for certain types of structure, such as portal frames.

The authors recommend the use of rigid-plastic analysis for hand calculations, using connections that have a configuration which is known to be sufficiently ductile. Because the plastic hinges form in the partial strength connections, the beam analysis is divorced from column considerations.

Table 1.1 summarises the characteristics that are needed for connections in frames designed using the various methods currently available. The method recommended in this publication (and explained in detail in Section 3) is shaded in the table.

Global analysis - SLS

Elastic analysis should be used to predict frame behaviour under serviceability loading. Because all composite connections complying with this guide may be assumed to be 'rigid', they may be modelled as fully continuous joints between frame members once composite action is achieved.

Connections will be considerably more flexible during construction, and this may influence some aspects of frame behaviour.

Table 1.1 General methods of ULS design

DESIGN		CONNECTIONS	COMMENTS
Type of Framing	**Global Analysis**	**Properties**	
Simple	Pin Joints	Nominally Pinned	Economic method for braced multi-storey frames. Connection design is for shear strength only (plus robustness requirements).
Continuous *(Note 1)*	Elastic	Rigid	Conventional elastic analysis.
	Plastic	Full Strength	Plastic hinges form in the adjacent member, **not** in the connections. Popular for portal frame designs.
	Elastic-Plastic	Full Strength and Rigid	
Semi-Continuous *(Note 2)*	Elastic	Semi-Rigid	Connections are modelled as rotational springs. Prediction of connection stiffness may present difficulties.
	Plastic	Partial Strength and Ductile	Wind-moment design is a variant of this method (for unbraced frames).
	Elastic-Plastic	Partial Strength and/or Semi-Rigid	Full connection properties are modelled in the analysis. Currently more of a research tool than a practical design method for most frames.

Note 1 BS 5950: Part 1 refers to these design methods as 'Rigid' and 'Semi-Rigid' respectively.
Note 2 Shading indicates the design method considered in Section 3 of this document.

1.6 CONNECTION CHARACTERISTICS

It should not be assumed that a moment connection, be it composite or not, is adequate simply because it is capable of resisting the bending moment, shear and axial forces predicted by a frame analysis. It is also necessary to consider either the rotational stiffness or the rotation capacity (ductility) of the connection, depending on the type of analysis adopted.

The characteristics of a connection can best be understood by considering its rotation under load. Rotation is the change in angle (ϕ) that takes place at the connection, as shown in Figure 1.4. The three important connection characteristics are illustrated in Figure 1.5. These three characteristics are:

- **Moment Capacity**
  The connection may be either *full strength* or *partial strength* (relative to the resistance of the composite beam in hogging), or *nominally pinned* (having negligible moment resistance).
- **Rotational Stiffness**
  The connection may be *rigid, semi-rigid* or *nominally pinned* (having negligible rotational stiffness).

- **Rotation Capacity**
  Connections may need to be *ductile*. This characteristic is less familiar than strength or stiffness to most designers, and is necessary when a connection needs to rotate plastically during the loading cycle. Considerable connection ductility may be needed if a frame is to be analysed plastically.

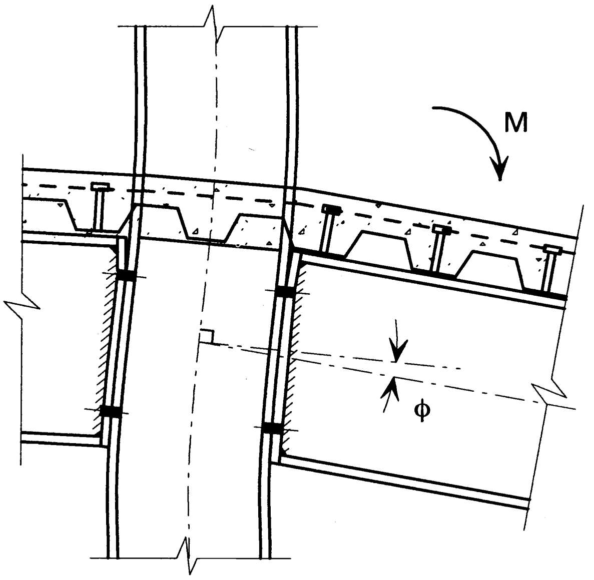


Figure 1.4 Rotation of a composite connection

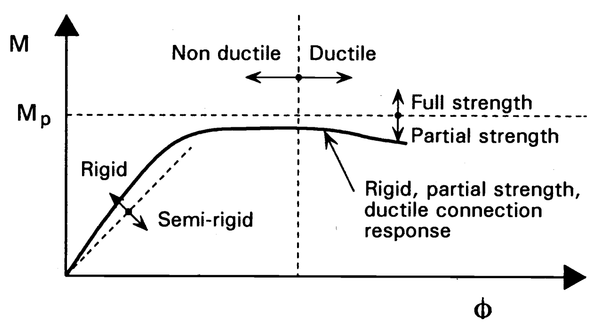


Figure 1.5 Classification of connections

Figure 1.5 shows boundaries between rigid/semi-rigid, full strength/partial strength, and non-ductile/ductile, in addition to a typical composite connection response. The typical curve indicates that composite connections are normally ductile, rigid, and partial strength.

Although Eurocode 4 (EC4) will present a method for calculating the stiffness of a composite end plate connection based on the approach of EC3, this can be a tortuous process. Assessing a connection's rotation capacity is also difficult, and the rotation required depends on parameters such as the loading arrangement and whether the frame is braced or unbraced.

In this guide a set of simple rule-of-thumb guidelines is presented to ensure that the frame design assumptions are not invalidated by the use of inappropriate connections. The designer has no need to consider explicitly either connection stiffness or connection ductility.

Any composite connection satisfying the detailing rules given in this guide (see Sections 4 and 5) may be assumed to be rigid once concreted. Elastic methods may therefore be used for frame analysis in the final state, with no need for the designer to determine the exact value of composite connection stiffness.

The rotation capacity of a composite connection may be less than that of the steel detail alone. One reason for this is that reinforcing steel is generally able to undergo less elongation before fracture than typical structural steel. The detailing requirements given in this guide ensure that, in terms of ductility, all composite connections will fall into one of two categories:

- 'compact' connections are sufficiently ductile to ensure that a stress block model can be used to predict the moment resistance
- 'plastic' connections have sufficient additional ductility to ensure that they can behave as a plastic hinge.

Detailing requirements (particularly concerning a minimum area of reinforcement) are more onerous for 'plastic' connections.

The standard connections presented in Section 6 are 'plastic', and may be used as such in either propped or unpropped braced frames analysed using plastic methods.

1.7 EXCHANGE OF INFORMATION

The design of a steel frame is often undertaken in two distinct stages, with the frame members designed by one engineer/organisation, and the connections designed by another. This method of working may be inappropriate when composite connections are used, because of interdependence of member and connection resistances, and because of the interaction between steel and concrete components.

It will often be prudent for the designer of the members in a semi-continuous frame to undertake the composite connection design, or at least to specify 'industry standard' details. Integrating member and connection design is particularly important for composite connections, because of the interaction between the steelwork components and the local slab reinforcement.

Care must be taken to ensure that requirements for any connections not designed by the member designer are clearly defined in the contract documents and on the design drawings. Connections that have been chosen to be composite should be clearly identified. The *National Structural Steelwork Specification for Building Construction*[9] gives appropriate guidance on the transfer of information.

In addition to the exchange of information at the design stage, the use of composite connections should be noted in the building owner's manual for future reference. Their presence may influence future modifications and demolition of the building.

1.8 MAJOR SYMBOLS

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

B	Width of section (subscript b or c refers to beam or column)
b_p	Width of plate
C	Compression force
D	Depth of section (subscript b or c refers to beam or column)
d	Depth of web between fillets *or* diameter of bolt
d_s	Depth of slab above decking
e	End distance
F	Force (subscript indicates whether in reinforcement, bolt etc)
f_{cu}	Cube strength of concrete
f_y	Yield strength of reinforcement
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 b or c refers to beam or column) *or* tension force
t_p	Thickness of plate
t	Thickness of web (subscript b or c refers to beam or column)
r	Root radius of section
V	Shear force
Z	Elastic modulus
γ_m	Material strength factor

Lengths and thicknesses stated without units are in millimetres

2 CONNECTION DESIGN

2.1 DESIGN PHILOSOPHY

The design model presented here uses the 'component approach' adopted in Eurocodes 3 and 4[7,10]. Using this approach, the moment capacity of the connection is determined by considering the strength of each relevant component, e.g. the tensile capacity of the slab reinforcement. Adopting a 'component approach' means that the designer can apply different aspects as appropriate to a particular situation. For example, the model can be applied to both composite and non-composite end plate connections. In the case of a non-composite connection, checks relating to the reinforcement are ignored. The Eurocode model has been validated by comparison with extensive test results.

The strength checks given in this document have been modified to conform with British standards eventhough the design philosophy is taken directly from the Eurocodes.

The connection resists moment by coupling tension in the reinforcement and upper bolts with compression in the lower part of the beam. The moment capacity is calculated by considering appropriate lever arms between the components. The force transfer mechanism is shown schematically in Figure 2.1. Evaluation of the tension and compression components that form a composite end plate connection are discussed in Sections 2.2 and 2.3.

The moment capacity of the connection may be evaluated by *plastic analysis*, using 'stress block' principles, provided that:

- There is an effective compression transfer to and through the column.
- The connection is detailed such that the maximum possible reinforcement *and* tension bolt capacities are generated. The reinforcement force is governed by yielding of the bars, whereas the bolt forces are governed by yielding of the end plate and/or column flange. This requires appropriate levels of strain to develop in the different tensile components (see Section 4).
- There are sufficient shear connectors to develop the tensile resistance of the reinforcement, with force being transferred through the concrete.
- The reinforcement is effectively anchored on both sides of the connection.
- Premature buckling failure, for example of the column web, is avoided.

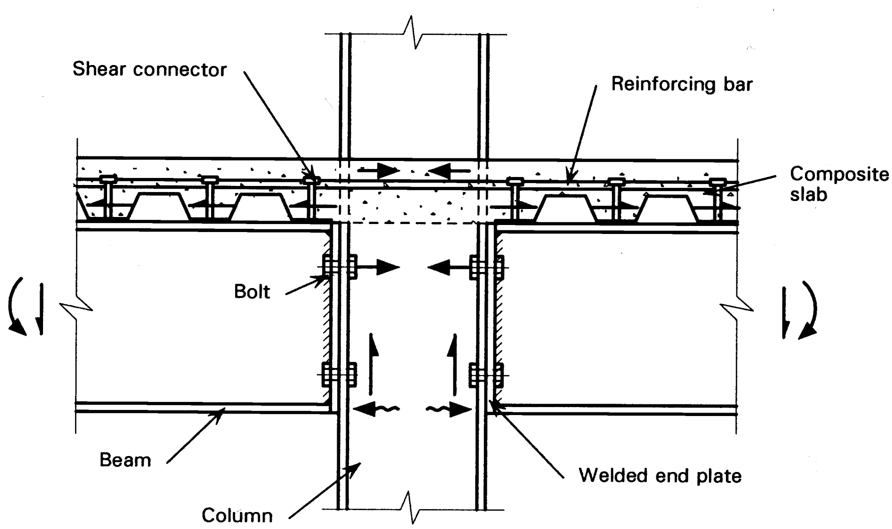


Figure 2.1 Typical composite connection at internal column, showing force transfer by the various elements

Tests show that, by the ultimate limit state, rotation has taken place with the centre of rotation in the lower part of the beam. For relatively small areas of reinforcement, compression is concentrated at the level of the centre of the lower beam flange. When the reinforcement area is greater and the compressive force is such that it exceeds the capacity of the beam flange, compression extends up into the beam web, and the centre of rotation changes accordingly.

2.2 TENSION COMPONENTS

Three of the connection components govern the magnitude of tensile force that can be generated. They are:

- the reinforcement
- the upper row(s) of bolts
- the interface shear connection in the hogging moment region of the beam.

2.2.1 Tensile force in the reinforcement

To effectively contribute to the connection behaviour, the reinforcement must be located within a certain distance of the centre line of the column (detailing rules are given in Section 5). This distance is *not the same* as the effective breadth of slab in the negative (hogging) moment region of the beam (as defined in Clause 4.6 of BS 5950: Part 3: Section 3.1[(5)]).

Tests and models[(8)] have shown that connection rotation capacity increases as the area of reinforcement increases. Minimum values of reinforcement area for 'compact' and 'plastic' connections are given in Section 4.

The maximum area of reinforcement in the connection is governed by:

- the ability of the shear connectors in the negative moment region to transfer the required force to the reinforcement (any excess reinforcement would simply be redundant)
- the compression resistance of the beam (any excess reinforcement would be redundant)
- the resistance of the column web, with due consideration of any stiffeners (any excess reinforcement could provoke a non-ductile failure)
- the strength of the concrete that bears against the column under unbalanced moment (any excess reinforcement could provoke non-ductile failure).

Connection moment capacity is calculated assuming that bar reinforcement yields. Adoption of the detailing rules given in Sections 4 and 5 will ensure that the steelwork components do not fail before the reinforcement has undergone sufficient strain to achieve this. This allows the contribution of the upper bolts to the connection moment capacity to be considered. The contribution to moment capacity of any mesh reinforcement that may be present in the slab should be ignored, because it fails at lower values of elongation than do larger bars.

2.2.2 Tensile force in the bolts

Although tension in the upper rows of bolts can be ignored to make a simple estimate of the connection resistance, the final connection design should consider their contribution. Ignoring the bolt forces underestimates the compression acting on the column, and could be unsafe if it led to a non-ductile compression failure. The design procedures given in Section 4 explain how to calculate the magnitude of the bolt forces.

The bolt row furthest from the beam compression flange tends to attract more tension than the lower bolts. Traditional practice in steelwork design has been to assume a triangular distribution of bolt row forces, based on a limit imposed by the furthest bolts. Although the method presented in Section 4 also gives greater priority to the upper bolts, 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 (as in a triangular distribution). Bolts near a point of stiffness, such as the beam flange or a stiffener, attract more load. Surplus force in one row of bolts can be transferred to an adjacent row that has a reserve of capacity. This principle is closer to the way connections perform in practice.

Bolt rows may only contribute their full capacity to the tensile resistance of a composite connection when the connection detailing is appropriate. In some situations, primarily when considerable reinforcement is present, a large compressive force in the beam means that the neutral axis is relatively high in the beam web. Deformations of the end plate and/or column web in the locality of bolt rows that are too close to the neutral axis will not be sufficient to develop the full bolt row forces that are associated with plate yielding. A simple linear reduction of bolt row forces is suggested in

Section 4.2 Step 1D for rows that are less than 200 mm from the neutral axis.

When plastic frame analysis is adopted, as is recommended in this guide, the connections must have a substantial rotation capacity. A composite connection with a steel detail that allows substantial deformation of the end plate to take place without bolt failure, and which has appropriate reinforcement detailing (see Sections 4 and 5), may be assumed to be 'plastic'; its rotation capacity will be sufficient for a plastic hinge to form at the connection. If deformation of the steelwork detail is not primarily limited to the end plate or column flange (known as 'mode 1' according to Reference 7), ductility cannot be assumed unless it has been demonstrated by testing.

2.2.3 Longitudinal shear force

The development of the full tensile force in the reinforcement depends on longitudinal shear force being transferred from the beam to the slab via the shear connectors and concrete, as shown in Figure 2.2. BS 5950: Part 3[5] requires that full shear connection is provided in the negative moment region. The need for full shear connection has been confirmed by tests on composite connections. The reinforcement should extend over the negative moment region of the span and be anchored a sufficient distance into the compression region of the slab to satisfy the requirements of BS 8110[11] (for example 40 times the bar diameter for a 'Type 2 deformed' bar in concrete with a cube strength of 30 N/mm^2).

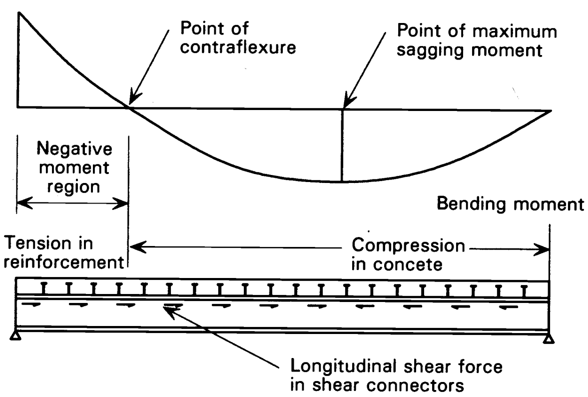


Figure 2.2 Transfer of longitudinal shear forces in a composite beam

2.3 COMPRESSION COMPONENTS

Two of the connection components govern the magnitude of compression force that can be accommodated. They are:

- the beam lower flange and adjacent web
- the column web.

2.3.1 Beam lower flange and web

Transfer of the compression force through the connection relies on direct bearing of the lower part of the beam on the column. To establish the depth of beam in compression, the designer should initially compare the applied compressive force with the resistance of the lower flange alone, assuming a design stress of $1.4p_y$. The factor of 1.4 allows for strain hardening and some dispersion into the web at the root of the section[4]. If the magnitude of applied compression does not exceed this flange resistance, the centre of compression should be taken as the mid-depth of the flange.

However, most composite connections will have substantial reinforcement, and the compression resistance required is likely to exceed the flange limit. In such cases compression is assumed to extend into the beam web, and then the resistance should be based on $1.2\ p_y$. An appropriate centre of compression must be adopted when calculating the moment capacity.

Because a plastic stress block model is considered, with design stresses in excess of yield, the design procedures are only appropriate for beams with flanges that are either Class 1 (plastic) or 2 (compact), and webs that are Class 1, 2 or 3 (semi-compact).

2.3.2 Column web

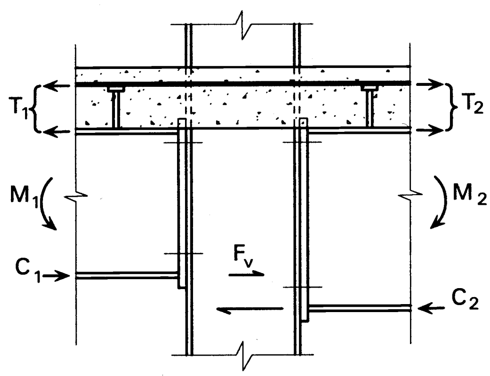
In addition to considering the resistance of the beam flange in compression, checks should be made on the local resistance of the column in compression. The buckling or crushing (bearing) resistance of the column web may limit the maximum compression force that can be transferred. This may be a particular problem for composite connections with a substantial area of reinforcement.

Because both crushing and buckling failure of the column web are non-ductile, they cannot be allowed to govern the moment capacity of a 'plastic' connection. Stiffeners must be added, or a heavier column section used, to avoid premature failure. Stiffener design is covered in Section 4.

Whilst column web compression stiffeners may often be hard to avoid in composite connections, their addition increases fabrication costs, and may complicate the positioning of orthogonal beams. A more economic solution might be to use a heavier column section that requires no stiffening. The presence of end plate connections on orthogonal beams connecting into the column web may prevent web buckling[4], but will not increase the column web crushing resistance. Supplementary web plates are an alternative form of web reinforcement that avoids clashes with other beams.

2.4 COLUMN WEB PANEL

For major axis connections, the column web panel must resist the horizontal shear forces. When checking panel shear, any connection to the opposite column flange must be taken into account, since the web must resist the resultant of the shears. In a one-sided connection with no axial force, the web panel shear F_v is equal to the compressive force in the lower part of the beam. For the case of a two-sided connection with balanced moments, the web panel shear is zero.



$F_v = C_2 - C_1$

Figure 2.3 Column web panel shear

2.5 VERTICAL SHEAR COMPONENTS

The vertical shear resistance of a connection relies on the steel components. Any shear resistance of the concrete or reinforcement should be ignored because of cracking in the slab. Traditionally, the lower bolts in a steel connection are assumed to resist the total applied shear force. However, although loaded in tension, the upper bolts may also resist a proportion of their design shear resistance (according to BS 5950: Part 1, Clause 6.3.6.3, the combined utilisation for shear and tension may be up to 1.4, so a bolt that is fully loaded in tension can still achieve 40% of its shear capacity).

Limited research[12] has suggested that the presence of high vertical shear force, or high axial load in the column, may reduce the moment capacity of a composite connection. However, in the absence of further information the authors do not feel it appropriate to include such considerations for composite connections used in orthodox frames with typical loading. For other cases, e.g. beams subject to heavy concentrated point loads near their supports, alternative considerations may be appropriate (see Reference 12).

2.6 STRUCTURAL INTEGRITY

It is a requirement of both the Building Regulations[13] and BS 5950: Part 1 [6] (Clause 2.4.5.2) that all building frames be effectively held together at each principal floor and roof level[7]. Steelwork details in accordance with this publication will generally be capable of carrying the basic 75 kN tying force[1] required by the British Standard (see Reference 1). However, larger tying forces may be required for tall, multi-storey buildings. The tensile capacity of the reinforcement (when properly anchored) may be added to the capacity of the bare steel connection in such situations. For the standard connection details, the resistance of the reinforcement and bolts may be extracted directly from the capacity tables in Appendix B.

The designer is advised that current rethinking of robustness requirements may lead to revised design criteria for structural integrity in the near future.

3 FRAME DESIGN

3.1 INTRODUCTION

This Section gives an overview of recommended analysis and design procedures for composite semi-continuous braced frames. Guidance is not given on checking of the bare steel frame under construction loading, which must however be considered as part of the frame design process.

3.2 ULTIMATE LIMIT STATE

The following recommended frame design procedures **may only be adopted when 'plastic' connections are used.** Plastic frame analysis and design is recommended because of its economy and simplicity.

Because plastic hinges are assumed to form in the connections rather than the adjacent beams, the composite connections must be partial strength (i.e. have a lower moment capacity than the beam in hogging). The standard connections presented in Section 6 are all partial strength. Connections must also be 'plastic' because the hinges are assumed to rotate. The standard connections presented in Section 6 are all 'plastic'. Despite the assumption of plastic hinge formation in the connections, it is not necessary to consider alternating plasticity in the connections, or incremental collapse of the frame[14].

The assumption that hinges form in the connections between the members allows the beams and columns to be considered separately, as discussed in Sections 3.2.1 and 3.2.2 respectively.

3.2.1 Beams

In a semi-continuous braced frame, beams are designed for a lower sagging moment than in an equivalent simple frame. This is possible because the connections allow hogging moments to be generated at the beam supports. The weight and/or depth of the beams can therefore be reduced. The influence of support moments on the required beam sagging moment capacity is illustrated in Figure 3.1, which shows moments for a beam that is:

(a) simply supported at both ends

(b) simply supported at one end and semi-continuous at the other

(c) semi-continuous at both ends.

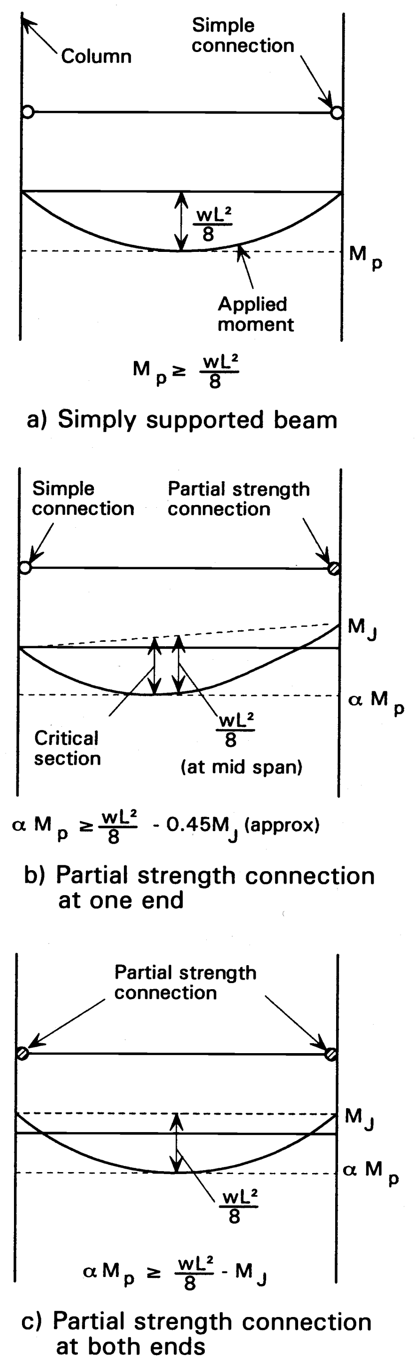


Figure 3.1 Applied moments and moment capacities for beams with different support conditions (UDL, α = 0.85)

Figure 3.1 shows schematically how the free bending moment ($wl^2/8$) is related to the moment capacities of the beam and connections for design. The benefit of semi-continuous construction in reducing the sagging moment that the beam must resist in a semi-continuous braced frame is evident. Despite the presence of the reduction factor α, the sagging moment is considerably reduced by the presence of moment resisting connections.

The beam sagging moment capacity should be determined using rules given in BS 5950: Part 3[5]. However, a reduction factor α must be applied to the moment capacity of the beam in sagging when connections detailed in accordance with this guide are used. The reduction factor is needed in order to limit the amount of plastic deformation that takes place in the beam in sagging, and thereby limit the required connection rotation[15]. Although the required reduction factor is a function of the load arrangement, the grade of steel, and whether or not the beam is propped during construction, a value of 0.85 may be used for all cases. A less conservative value could be used in some cases, but a single value is proposed for simplicity.

In addition to using a reduced effective beam moment capacity, to further limit support rotation requirements the connection moment capacity must exceed 30% of the beam moment capacity in sagging (this is achieved by all the standard connections given in Section 6). A lower limit on connection strength is necessary because connection rotation requirements decrease as the relative strength of the connection increases, tending towards zero for a beam with ends that are fully built-in.

A third requirement in order to ensure that required connection rotations are not excessive is that the span to depth ratios of beams must satisfy the following limits[15] (where D is the *total* depth of steel beam plus slab):

- $L/D \leq 25$ for beams subject to UDL, multiple point loads or a central point load
- $L/D \leq 20$ for beams subject to two point loads (at third span points).

Combined connection and beam strengths that can be achieved using the standard connection details are presented in Table 3.1. This table can be used at the scheme design stage to identify possible beam sizes, as illustrated by the following example. For a beam subject to UDL, the free bending moment $wl^2/8$ should be compared with values of M_{TMIN} or M_{TMAX}. As an example, consider the following parameters:

Beam span	10 m
Dead load	4 kN/m^2
Imposed load	6 kN/m^2
Loaded width	3 m

Total factored load = 45.6 kN/m
Free bending moment is $wl^2/8$ = 570 kNm

From Table 3.1, possible beams would include:
$356 \times 171 \times 45$, S275 (which can support a moment in the range 558 to 583 kNm)
$305 \times 127 \times 48$, S275 (531 to 595 kNm)
$254 \times 146 \times 43$, S355 (558 to 606 kNm)

If the beam were simply supported, the free bending moment would be compared with M_p. The shallowest simply supported composite beam that could satisfy the input parameters given above would be a $356 \times 171 \times 67$ (S275). Comparison of this beam size with the possible semi-continuous solutions listed above clearly shows the benefits of using composite connections.

A recommended procedure for beam design is summarised in Figure 3.2. This flow chart is based on the following assumptions:

- preliminary studies have been carried out that show that composite connections are worthwhile for the beam in question
- column orientations have been identified
- preliminary column sizes are known
- the final choice of column size will depend on the connection details that will be chosen.

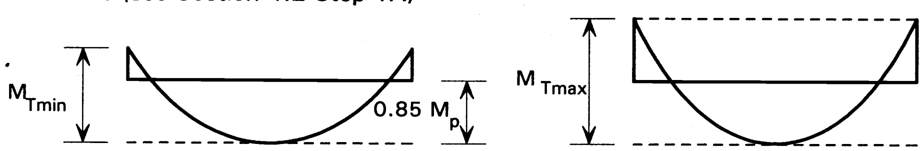
Table 3.1 Combined beam and connection moment capacity ranges

BEAM Serial Size	S275			S355		
	BEAM 0.85 M_p	BEAM + CONNECTION		BEAM 0.85 M_p	BEAM + CONNECTION	
		M_{TMIN}	M_{TMAX}		M_{TMIN}	M_{TMAX}
533x210x122	1200	1563	2238	1558	2133	2596
109	1063	1423	1939	1383	1954	2438
101	980	1338	1780	1277	1846	2258
92	920	1277	1636	1187	1753	2071
82	822	1176	1445	1036	1599	1784
457x191x98	883	1151	1619	1148	1510	2056
89	796	1062	1444	1036	1395	1838
82	752	1017	1348	969	1327	1674
74	677	941	1187	875	1231	1526
67	604	866	1050	778	1132	1330
457x152x82	716	983	1316	932	1293	1677
74	642	908	1171	837	1196	1485
67	596	860	1047	769	1125	1345
60	528	791	914	682	1037	1161
52	449	710	449	580	932	984
406x178x74	650	889	1162	839	1078	1465
67	582	819	985	752	989	1285
60	519	755	888	671	907	1130
54	456	690	780	589	823	988
406x140x46	384	619	647	496	731	824
39	313	313	313	405	637	660
356x171x67	550	761	978	710	921	1174
57	459	668	780	593	802	1002
51	406	613	697	525	732	880
45	352	558	583	453	659	742
356x127x39	299	502	502	386	589	621
33	246	246	246	319	521	319
305x165x54	417	599	697	537	719	896
46	353	533	599	456	636	734
40	306	485	499	396	575	640
305x127x48	349	531	595	451	633	759
42	303	484	528	390	571	615
37	265	436	436	344	515	582
305x102x33	236	393	399	306	469	469
28	199	342	199	256	399	424
25	168	168	168	217	217	217
254x146x43	316	467	515	407	558	606
37	269	418	448	348	497	527
31	219	347	347	253	381	381
254x102x28	195	316	216	251	372	372
25	170	170	170	219	219	219
22	144	144	144	186	186	186

M_p values are based on typical composite beam details, assuming a slab depth of 120 mm, and full shear connection[5]

M_{TMIN} is the sum of 0.85 M_p and the connection moment capacity with minimum reinforcement

M_{TMAX} is the sum of 0.85 M_p and the connection moment capacity with maximum reinforcement. Note that maximum reinforcement limitations for a particular case may prohibit attainment of this value (see Section 4.2 Step 1A)



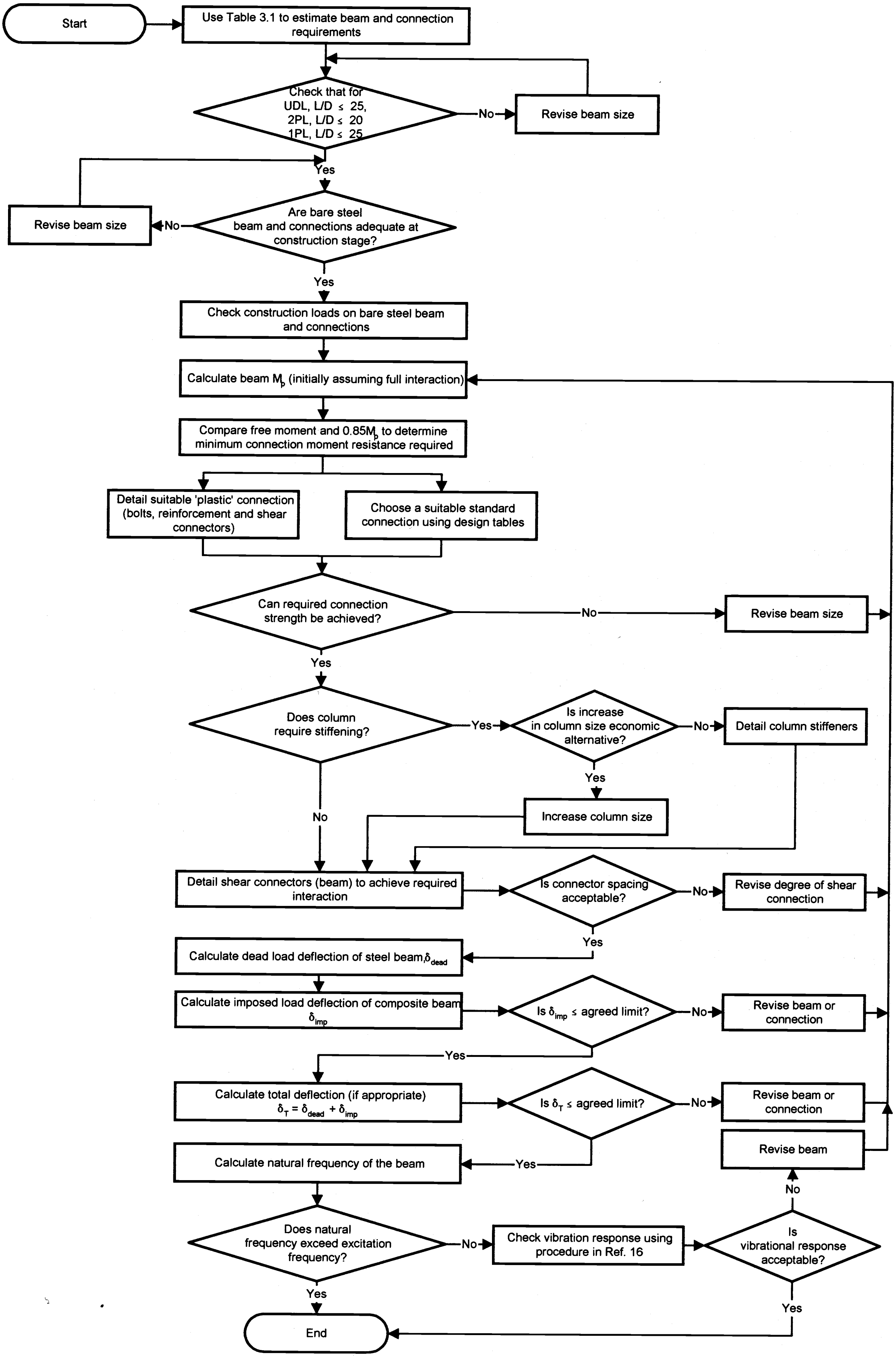


Figure 3.2 Detailed design procedure for a beam with composite connections in a semi-continuous braced frame

Lateral buckling adjacent to supports

When the steelwork connections are required to transfer moment at the construction stage, it may be necessary to check the stability of the bottom flange under the worst case loading of the wet weight of concrete on one span, which can cause hogging moment over a significant portion of the adjacent span. The lateral stability of the steel beam should be checked in accordance with BS 5950: Part 1[(6)], taking account of moment variation along the beam. The top flange may be assumed to be laterally restrained either by transverse beams or by the steel decking, which must be properly fixed in position.

At the composite stage, lateral torsional buckling of the section is prevented in the sagging moment region by attachment to the slab. In the negative moment region, the compression flange cannot displace laterally without transverse bending of the web. This is known as lateral distortional buckling. The critical parameter is the *D/t* ratio of the web; the higher the ratio the greater the tendency for buckling.

EC4[(10)] offers a general design procedure, but for sections up to 550 mm deep (S275) or 400 mm deep (S355), it states that no checks are required on the stability of the lower flange provided certain loading and detailing requirements are respected (EC4 Clause 4.6.2). Additional guidance may be found in Reference 17. For deeper beams, the designer should refer to procedures given in EC4 Annex E[(10)], and consider an applied negative moment equal to the resistance of the connection.

In practice, for any depth of beam a check of lateral stability in the hogging moment region is only needed when connections possess a resistance in excess of approximately 80% of the moment capacity of the composite beam in hogging. The capacity tables in Section 6 highlight connections that may possess a capacity in excess of this limit. It should be noted however that in the interests of simplicity, the highlighting is based on a comparison of the connection moment capacity with that of a grade S275 beam. If a grade S355 beam were used, the relative connection capacity would clearly decrease, below the 80% limit in some cases. The designer should calculate the beam capacity, for comparison, when necessary.

3.2.2 Columns

The moment capacity of a composite connection is generally low in comparison to that of the composite beam in sagging. Because of this, when beams are **propped** during construction, the composite connection moment capacity is normally developed under dead load alone when the props are removed. There is therefore no increase in support moments as imposed load is applied, the connection merely rotates, and frame moments under pattern loading are the same as when imposed load is present on all beams. Column checks therefore need only be performed for the 'all spans loaded' case. For propped construction, this means that internal columns need only be designed to resist moments that are due to differing connection strengths either side of a node. Moments due to eccentric beam reactions (acting at either the column face or the face plus 100 mm nominal eccentricity, as considered in simple design to BS 5950: Part 1) need not be added to the connection moment capacities. Beams should therefore be considered as spanning between the centrelines of the columns.

When **unpropped** construction is adopted, the connections only act compositely under imposed load. Under dead load, only the moment capacity of the bare steel connection can be mobilised. Pattern loading therefore gives rise to unbalanced moments on the column even when opposing composite connections of equal moment capacity are used.

The designer may conservatively assume that the magnitude of the unbalanced moment is equal to the difference between the composite connection capacity on one side of the column and the bare steel capacity on the other. The unbalanced moment should be distributed between the column lengths above and below the node, in proportion to the stiffness (*I/L*) of each length. This procedure is as for BS 5950: Part 1 *columns in simple construction.*

For a less conservative calculation of column moments with unpropped construction, the designer should redistribute the unbalanced beam end moments considering an elastic sub-frame, as proposed in BS 5950: Part 1 Clause 5.6.4. The elastic beam stiffness is a function of the composite beam properties in both hogging and sagging. A global value equal to 1.8 times the second moment of area of the bare steel section (which can be obtained from standard section tables) which may be conservatively adopted. Connection stiffness need not be incorporated into the sub-frame model because the connections are 'rigid' in their composite state. The moments that can be distributed to the beams are, however, limited by the composite connection moment capacities. Any excess beam moments that are predicted by an elastic sub-frame distribution should be reallocated to the column.

When choosing column effective lengths, the designer must assess the degree of restraint that will be offered by beams, column continuity, and/or base details. Appropriate general procedures are given in Appendix E of BS 5950: Part 1[(6)]. Although composite connections satisfying the detailing requirements given in this guide have an initial stiffness that is at least as great as that of conventional 'rigid' steel connections, the rotational restraint provided by some beams may only be comparable to that of a simply supported beam (because of plastic hinge formation at relatively low levels of load). The designer should pay particular attention to minor axis connections, which will normally be relatively flexible, and minor axis conditions will often dictate column size.

Further guidance on the choice of column effective lengths may be found in Reference 18.

3.3 SERVICEABILITY LIMIT STATE

Elastic analysis must be used to check frame behaviour under serviceability loading.

For checks on imposed load deflections, which should be based on the initial stiffness of the connections, any composite connection complying with the detailing rules given in this guide may be assumed to be 'rigid' when used in a braced frame. This assumption is based on experimental evidence. Full continuity between the frame members can therefore be assumed.

Alternatively, the composite beams may be assumed to be continuous over 'knife-edge' supports; procedures given in BS 5950: Part 3 clause 6.1.3[(5)] should then be used to calculate imposed load deflections. Support moments must be limited to the moment capacity of the connections. Simplified procedures for modelling the influence of pattern loading and shakedown are included in the Code.

If it is necessary to check total load deflections, for example to avoid excess ponding of the concrete during construction for an unpropped beam, it should be noted that the dead load deflection will depend on the stiffness of the connections at the stage when dead load is applied to the beam. This varies according to the steelwork detailing and construction procedure. When beams are unpropped, dead load deflections are a function of the bare steel connection properties. Guidance on the calculation of deflections in the bare steel state may be found in Reference 19.

4 "STEP BY STEP" DESIGN PROCEDURES

4.1 INTRODUCTION

This Section presents design procedures for composite connections subject to hogging moments (with the reinforced slab in tension). Composite connections subject to sagging moments, as may occur in some unbraced frames when wind loads are relatively high, behave in a different way. They are not covered by this publication as there is currently insufficient information available to develop a design model. The following design procedures are therefore only applicable to composite connections in braced frames.

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

- as a source of reference,
- for use in writing software,
- for use in checking output from software,
- for spreadsheet implementation.

The procedures were used to calculate moment capacities for a range of standard beam-to-column details, and these are presented in Design Tables in Appendix B.

The reader who is familiar with *Joints in steel construction: moment connections*[4] will note that the procedures for the steelwork component resistances are essentially unchanged. More explanation of some of these procedures may be found in Reference 4.

4.2 BEAM-TO-COLUMN CONNECTIONS

The procedures that follow are suitable for beam-to-column connections using flush end plates. Although all the checks are needed for (major axis) connections to column flanges, the following steps are not relevant for (minor axis) connections to column webs:

- Step 1B - column flange bending
- Step 1C - column web tension
- Step 2A - column web compression
- Step 3 - column panel shear
- Step 6 - design of stiffeners

Alternative checks for major and minor axis connections are given in Step 5.

Opposing beams connecting into a column web should be treated as semi-continuous over a 'knife edge' support, with the presence of the column web assumed to have no influence on beam behaviour. However, beams must be of similar depth to achieve continuity, because the webs of typical columns do not have sufficient stiffness and strength to transfer 'eccentric' compression forces. It is recommended that opposing beams are of the same serial size. Similar consideration must be given to beam-to-beam connections.

The sequence of design checks is presented in the form of a flow chart in Figure 4.2, and the zones considered in the steps are illustrated in Figure 4.1. A worksheet is included in Step 1 so that the process of calculating the reinforcement and bolt row forces can be set down in tabular form (see page 42).

A worked example illustrating the design of a connection using these procedures is given in Appendix A.

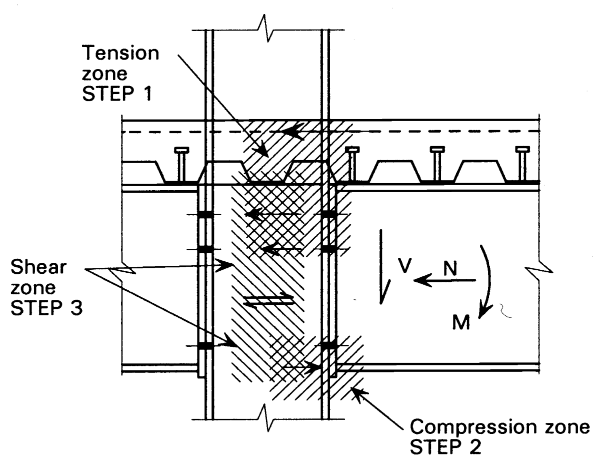


Figure 4.1 Check zones for a composite end plate beam-to-column connection

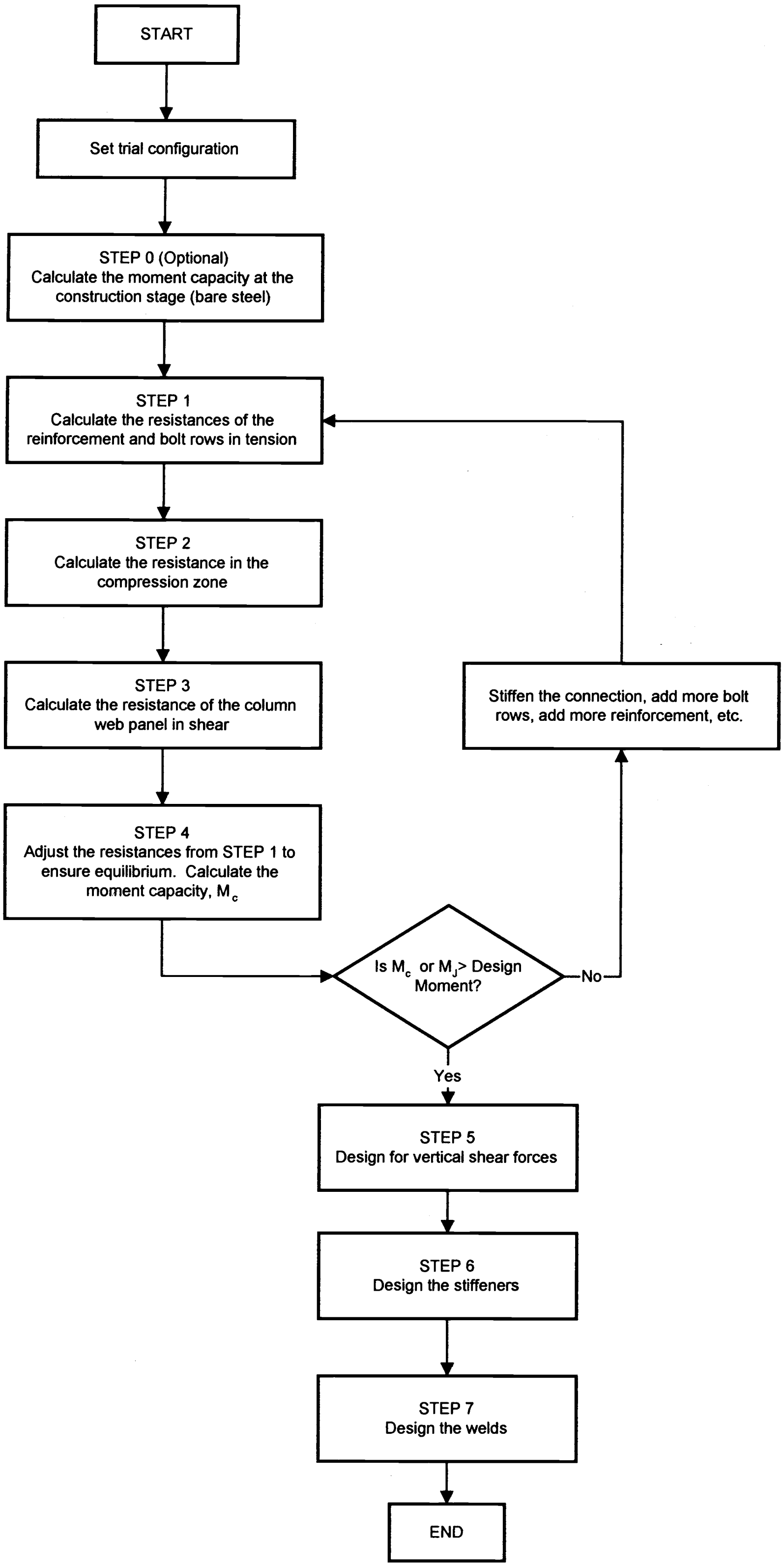


Figure 4.2 Flow diagram - connection design checks

CALCULATION OF MOMENT CAPACITY - CONSTRUCTION STAGE

It is important that the designer considers the bare steel performance under construction loading as part of the composite beam design. Indeed, a composite beam cannot be said to have been properly designed if the construction stage has not been considered.

If the moment capacity of the bare steel connection is needed for the construction stage checks, it should be calculated using the procedures given in Section 2 of Reference 4.

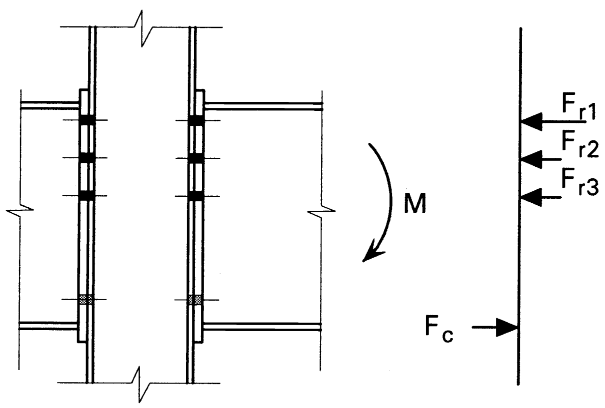


Figure 4.3 Moment resisting bare steel connection

STEP 1 POTENTIAL RESISTANCES OF REINFORCEMENT AND BOLT ROWS IN THE TENSION ZONE

The resistances of the tension components that are calculated first are only *potential* values. It may be necessary to reduce them in Step 4 in order to achieve equilibrium depending on the resistance of the connection compression components.

Reinforcement

The potential resistance of the reinforcement is limited by yielding of the bars, and by limitations on the minimum and maximum area of reinforcement that can be used. Details are given in Step 1A.

Bolts

The potential resistance of 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 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 bolt row is checked first in isolation and then in combination with successive rows above it, i.e.:

P_{r1} = (resistance of row 1 alone)

P_{r2} = Min. of:
resistance of row 2 alone
(resistance of rows 2 + 1) - P_{r1}

P_{r3} = Min. of:
resistance of row 3 alone
(resistance of rows 3 + 2) - P_{r2}
(resistance of rows 3 + 2 + 1) - P_{r2} - P_{r1}

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

For each of these checks, the resistance of a bolt row or a group of bolt rows is taken as the least of the following four values:

- Column flange bending/bolt yielding (Step 1B)
- End plate bending/bolt yielding (Step 1B)
- Column web tension (Step 1C)
- Beam web tension (Step 1C)

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

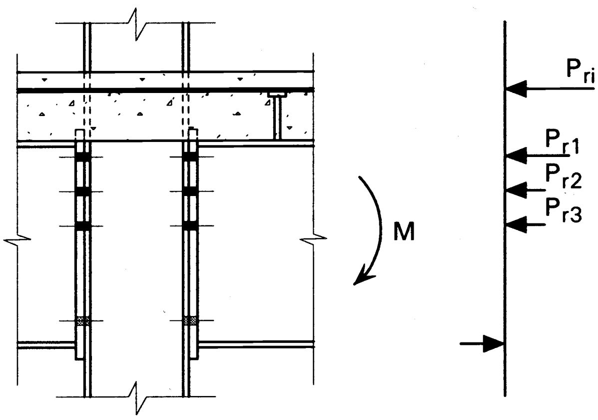


Figure 4.4 Potential resistance of reinforcement and bolt rows

STEP 1A REINFORCEMENT YIELDING

Potential resistance

The potential resistance of the reinforcement in tension is given by:

$$P_{reinf} = \frac{f_y A_{reinf}}{\gamma_m} \quad (1.1)$$

f_y = design yield strength of reinforcement

A_{reinf} = area of reinforcement within the effective width of slab for the connection (see Figure 4.6 and Section 5).

γ_m = partial safety factor for reinforcement (taken as 1.05)

Detailing rules for the reinforcement are given in Section 5.

Minimum area of reinforcement

In general, the rotation capacity of a connection increases as the area of reinforcement increases. This is because the level of strain at which the reinforcement fails, allowing for tension stiffening, increases[15]. A minimum area is therefore needed to ensure that 'compact' connections can undergo sufficient rotation to strain the reinforcement to yield (as is assumed in the moment capacity model). Minimum areas of reinforcement that should be provided in a 'compact' or 'plastic' composite connection are given in Table 4.1 as a function of:

- beam size
- beam steel grade
- reinforcement properties
- connection type - 'compact' or 'plastic'

The minimum reinforcement limits in Table 4.1 are more onerous for 'plastic' connections, because in addition to the need for yielding of the reinforcement, the connection must have sufficient rotation capacity to behave as a plastic hinge.

In Table 4.1 the minimum values, marked '5%', should normally be used. These values are appropriate for high yield bars complying with current British Standard BS 4449, grade 460B[20]. This grade of reinforcement has a mandatory requirement of 14% minimum elongation at fracture, and a non-mandatory requirement of 5% minimum elongation at maximum force. Grade B500B bars complying with BS EN 10080[21] are required to have similar properties; they must be able to achieve 5% total elongation at maximum force. Elongation at fracture and total elongation at maximum force are illustrated in Figure 4.5.

Minimum reinforcement areas are also given in Table 4.1 for connections that use reinforcement which is capable of achieving 10% minimum elongation at maximum force. The increased reinforcement ductility offers considerable advantages in some cases, because it permits the use of less reinforcement.

It is essential that when a design is based on the use of 10% elongation bars this is made clear in the project specification. This can be done by giving the bars an 'X' designation, rather than the 'T' generally used for high tensile bars[22]. The 'X' informs the contractor that the bars need specific, non-standard properties. It is recommended that, if possible, the reinforcement supplier uses coloured labels to clearly distinguish the high elongation 'X' bars on site. In case of doubt concerning the elongation capacity of bars, approximately half the UK manufacturers provide reinforcement suppliers with appropriate test information. It should therefore be relatively easy for the contractor to confirm suitability with his reinforcement supplier.

Bars that are currently produced in the UK using a hot forming process may be assumed to be appropriate for use with the '10%' limits. All 20 mm diameter bars produced by major manufacturers in the UK currently are hot formed, as are, often, 16 mm bars.

STEP 1A REINFORCEMENT YIELDING (continued)

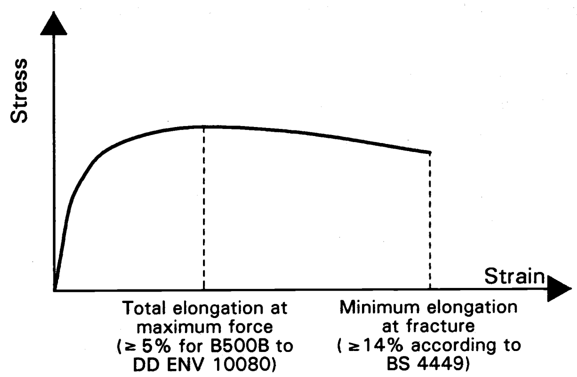


Figure 4.5 Elongation limits for reinforcement

Table 4.1 Minimum area of reinforcement - 'compact' and 'plastic' connections

Type	Steel	Rebar Elongation Limit	Beam Depth (mm)							
			203	254	305	356	406	457	533	610
Compact	S275	5%	500	500	500	500	500	600	750	1150
		10%	500	500	500	500	500	550	650	800
	S355	5%	500	500	500	500	500	600	750	1150
		10%	500	500	500	500	500	550	650	800
Plastic	S275	5%	500	500	500	650	1100	1450	1800	3000
		10%	500	500	500	500	500	600	750	1150
	S355	5%	500	500	600	1400	2100	3100	-	-
		10%	500	500	500	500	650	900	2000	2850

Note: A dash (-) in the table indicates that excessive reinforcement is required, options are not therefore practical

STEP 1A REINFORCEMENT YIELDING (continued)

Maximum area of reinforcement

The reinforcement area must also be limited to a maximum value in order to:

- prevent local concrete crushing failure under unbalanced loading
- keep the compression zone in the lower half of the steel beam.

The reasons for these limits are discussed below.

To consider potential concrete crushing failure, a truss model has been developed to represent how double sided composite connections behave when the applied moments on either side are unequal[8]. Figure 4.6 illustrates the components in the truss, showing that the connection resistance relies on the ability of the concrete to bear against the column on the low moment side. The net force in the reinforcement is therefore limited by the strength and area of concrete in bearing. An enhancement factor may be applied to the concrete strength because of its confinement[23].

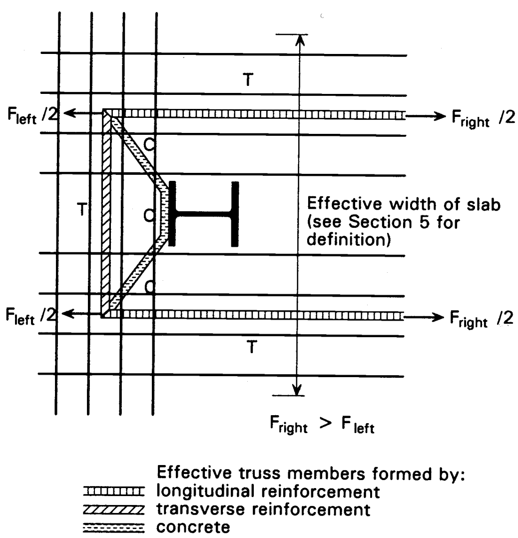


Figure 4.6 Truss model for connection behaviour under unbalanced moment

According to the truss model, the area of longitudinal reinforcement must not exceed:

$$A_L \leq \frac{0.6\, b_c\, d_s}{\mu} \frac{f_{cu}}{f_y} \qquad (1.2)$$

where:

b_c = width of column

d_s = depth of slab above decking

f_{cu} = cube strength of concrete

f_y = yield strength of the rebar

μ is a function of the difference in applied moments, and beam depths, either side of the node:

$$\mu = 1 - \frac{M_{low}}{M_{high}} \frac{h_{r1}}{h_{r2}} \qquad (1.3)$$

where:

M_{low} = the smaller applied moment (may be taken as the smaller connection moment capacity when 'plastic' connections are used)

M_{high} = the larger applied moment (may be taken as the larger connection moment capacity when 'plastic' connections are used)

h_{r1} = reinforcement lever arm on the high moment side

h_{r2} = reinforcement lever arm on the low moment side

In general, unbalanced moments will not be excessive in braced frames, so maximum reinforcement area limitations should not be restrictive. In a symmetric situation μ is zero, meaning that there is no upper limit on reinforcement area implied by this model.

Transverse reinforcement acts as a tension member in the truss model (see Figure 4.6). The area of transverse reinforcement must satisfy the following limit:

$$A_T \geq \frac{0.35\, \mu\, A_L}{\left(\frac{e_T}{e_L} - 0.3\right)} \qquad (1.4)$$

STEP 1A REINFORCEMENT YIELDING (continued)

where:

e_L ≈ $2.0b_c$ (this is the outer limit of the longitudinal reinforcement from the column centre line)

e_T ≈ $3.0b_c$

e_L and e_T are identified in Figure 5.1.

It is assumed that the longitudinal and transverse reinforcing bars have the same nominal yield strength. When 'plastic' connections are used, the transverse reinforcement area will only be approximately one tenth of the longitudinal reinforcement area, and BS 8110[11] minimum percentage limits may govern the area of transverse reinforcement.

In theory, the length of transverse reinforcement must be limited so that, whilst sufficient anchorage is provided for the bars to act in the truss, they do not affect the behaviour of the 'transverse beam connections'. In practice, this should not be critical. Detailing rules are given in Section 5.

To ensure adequate strain in the reinforcement, compression must be restricted to the lower half of the steel beam (i.e. the plastic neutral axis must not be higher than the mid-depth of the web).

Another possible reason for limiting the reinforcement area is to avoid the need for column compression stiffeners. For information, Table 4.2 indicates maximum values of reinforcement area that can be used, in combination with different numbers of tension bolts, before column compression stiffeners become necessary. Values are given for column sizes generally used in building, considering both S275 and S355 steel. The limits are based on the application of design Step 2A. This table may be used to assess the relative economics of different options, remembering that column stiffening is expensive.

Table 4.2 (a)	**Maximum areas of reinforcement that can be adopted without column compression stiffening, S275 columns**			
Column serial size	**Allowable area of reinforcement (mm^2)**			
	1 row M20	**2 rows M20**	**1 row M24**	**2 rows M24**
356x368x202	2160	1851	1936	1413
356x368x177	1664	1356	1440	918
356x368x153	1271	963	1048	525
356x368x129	906	598	683	160
305x305x198	2783	2474	2559	2036
305x305x158	1897	1589	1673	1150
305x305x137	1477	1169	1253	730
305x305x118	1105	797	881	358
305x305x97	788	479	564	41
254x254x167	2659	2351	2436	1913
254x254x132	1783	1475	1559	1036
254x254x107	1223	915	1000	477
254x254x89	797	488	573	50
254x254x73	520	212	297	0
203x203x86	1125	817	902	379
203x203x71	694	386	470	0
203x203x60	530	221	306	0
203x203x52	347	39	123	0
203x203x46	240	0	16	0
Note:	4x16 mm bars = 804 mm^2 6x16 mm bars = 1210 mm^2 8x16 mm bars = 1610 mm^2 10x16 mm bars = 2010 mm^2		4x20 mm bars = 1260 mm^2 6x20 mm bars = 1890 mm^2 8x02 mm bars = 2510 mm^2 10x20 mm bars = 3140 mm^2	
Values are based on γ_m = 1.05 and f_{sk} = 460 N/mm^2				

Table 4.2 (b) Maximum areas of reinforcement that can be adopted without column compression stiffening, S355 columns				
Column serial size	Allowable area of reinforcement (mm^2)			
	1 row M20	2 rows M20	1 row M24	2 rows M24
356x368x202	2926	2618	2703	2180
356x368x177	2287	1979	2063	1541
356x368x153	1780	1472	1557	1034
356x368x129	1324	1016	1100	578
305x305x198	3730	3422	3506	2983
305x305x158	2586	2278	2363	1840
305x305x137	2045	1737	1822	1299
305x305x118	1564	1255	1340	817
305x305x97	1153	845	929	406
254x254x167	3570	3262	3346	2824
254x254x132	2440	2132	2216	1694
254x254x107	1717	1408	1493	970
254x254x89	1180	872	956	434
254x254x73	810	502	587	64
203x203x86	1591	1283	1367	845
203x203x71	1045	737	822	299
203x203x60	822	514	598	75
203x203x52	584	276	361	0
203x203x46	447	139	224	0
Note:	4x16 mm bars = 804 mm^2 6x16 mm bars = 1210 mm^2 8x16 mm bars = 1610 mm^2 10x16 mm bars = 2010 mm^2		4x20 mm bars = 1260 mm^2 6x20 mm bars = 1890 mm^2 8x02 mm bars = 2510 mm^2 10x20 mm bars = 3140 mm^2	
Values are based on γ_m = 1.05 and f_{sk} = 460 N/mm^2				

STEP 1B END PLATE OR COLUMN FLANGE BENDING AND/OR BOLT YIELDING

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

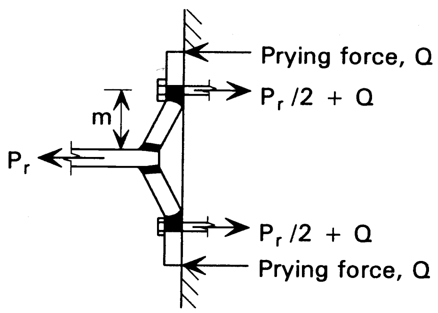
The potential resistance in tension of the column flange or end plate, P_r , is taken as the minimum value obtained from the three Equations *(1.5)*, *(1.6)* and *(1.7)*, below.

Note that:

- when mode 1 governs, connection behaviour is always ductile,
- when mode 2 governs, connection ductility should be demonstrated by testing,
- when mode 3 governs, connection behaviour is always non-ductile.

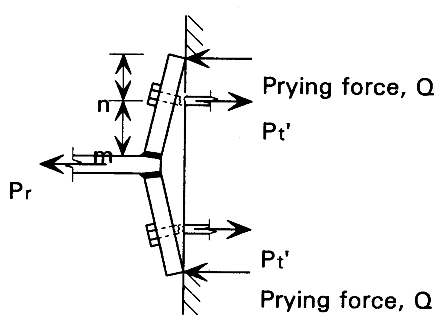
Mode 1 Complete flange or end plate yielding

$$P_r = \frac{4M_p}{m} \qquad (1.5)$$



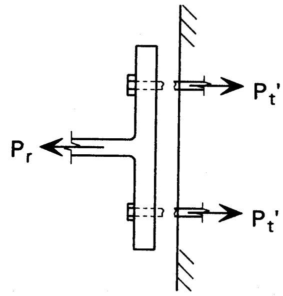
Mode 2 Bolt failure with flange/end plate yielding

$$P_r = \frac{2M_p + n(\Sigma P_t')}{m+n} \qquad (1.6)$$



Mode 3 Bolt failure

$$P_r = \Sigma P'_t \qquad (1.7)$$



where:

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 4.3)

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

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

$$= \frac{L_{eff} \times t^2 \times p_y}{4} \qquad (1.8)$$

L_{eff} = effective length of yield line in equivalent T-stub (see Tables 4.4, 4.5 and 4.6).

t = column flange or end plate thickness

p_y = design strength of column/end plate

m = distance from bolt centre to 20% distance into column root or end plate weld (see Figure 4.8)

n = effective edge distance (see Figure 4.8)

STEP 1B (continued)

Table 4.3 Tensile capacity of a single 8.8 bolt

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

Notes
1. 450 N/mm^2 is the tension strength of bolt according to BS 59590: Part 1
2. 560 N/mm^2 is the enhanced tension strength that may be used when prying action is considered explicitly (see Reference 4)

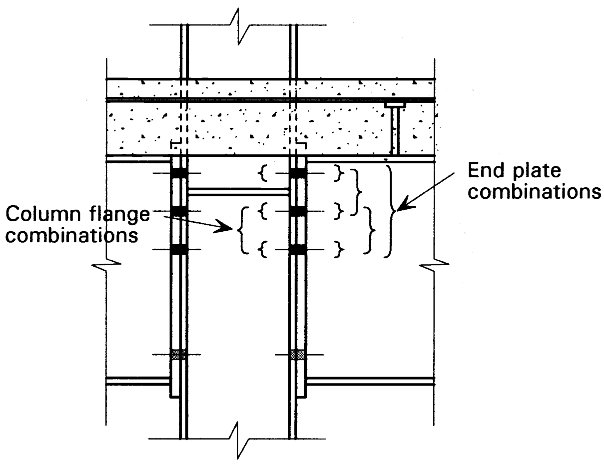


Figure 4.7 Influence of stiffeners

Backing Plates

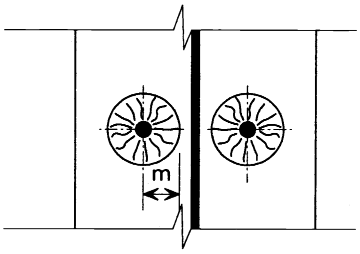
For small section columns with thin flanges, loose backing plates can increase the resistance of the column flange by preventing a Mode 1 bending failure of the flange. Design rules for backing plates are given in Step 6B.

Stiffeners

For end plate or column flange bending, bolt groups must be considered separately between stiffeners or the beam flange, as shown in Figure 4.7. The yield pattern of any bolt row below a stiffener (or flange) cannot combine with any row(s) above the stiffener/flange on the side where the stiffener/flange is.

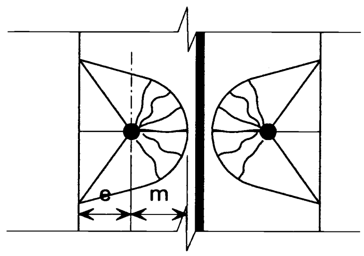
Table 4.4 L_{eff} for equivalent T-stubs for bolt row acting alone

Pair of bolts separated by a web in a column flange or end plate



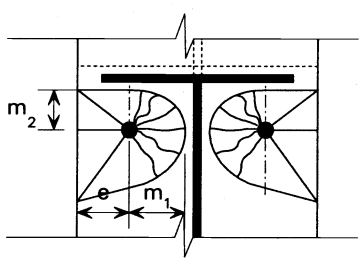
Pattern (i)
Circular yielding

L_{eff} = 2πm



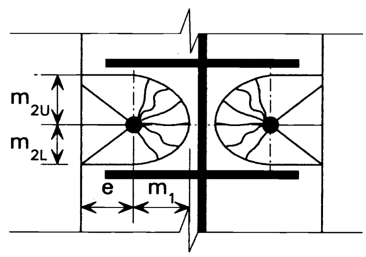
Pattern (ii)
Side yielding

L_{eff} = 4 m + 1.25e



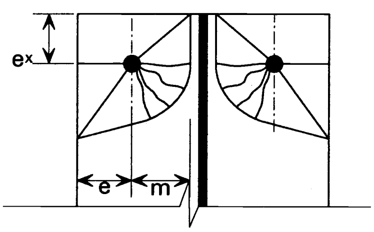
Pattern (iii)
Side yielding near beam flange or a stiffener

L_{eff} = αm_1



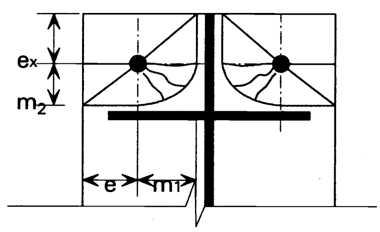
Pattern (iv)
Side yielding between two stiffeners

L_{eff} = αm_1 + $\alpha' m_1$ - ($4m_1$ + 1.25e)
(α is calculated using m_{2u}
α' is calculated using m_{2L})



Pattern (v)
Corner yielding

L_{eff} = 2m + 0.625*e* + e_x



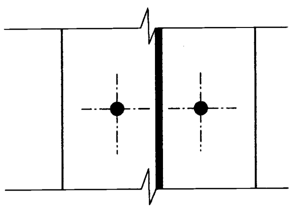
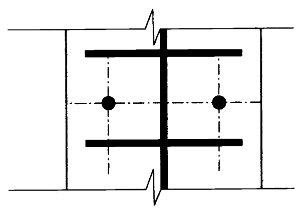
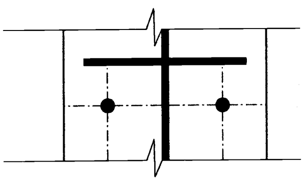
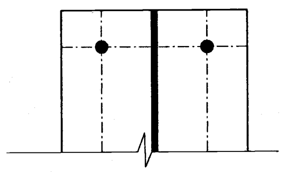
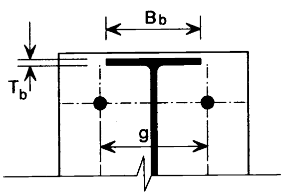
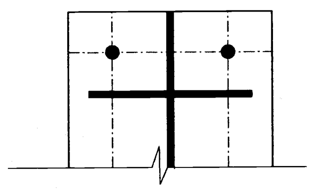
Pattern (vi)
Corner yielding near a stiffener

L_{eff} = αm_1 - ($2m_1$ + 0.625e) + e_x

Notes:

- Table 4.5 shows which of the above expressions have to be considered.
- Table 4.6 shows which parts of the above expressions have to be combined when bolt rows act as a group.
- Dimensions *m, e,* e_x are shown in Figure 4.8.
- The value of α is determined from the chart in Figure 4.9.
- L_{eff} is the length of the equivalent T-stub, **not** the length of the pattern shown.

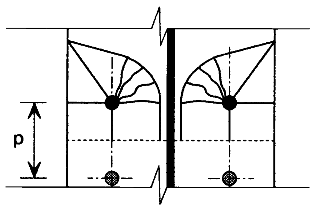
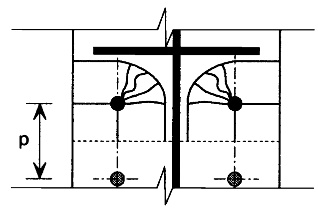
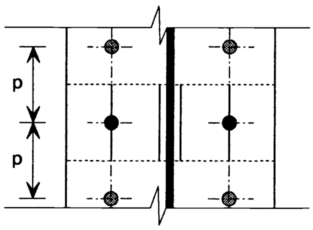
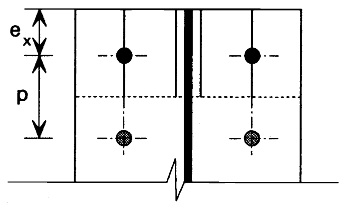
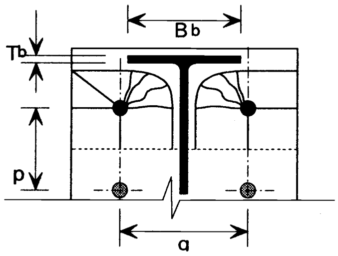
Table 4.5 L_{eff} to be considered for a bolt row acting alone (expressions to be used from Table 4.4)

	**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** Use: Min{Max{ii, iii},I}		**Bolt row next to a free end** Use: Min {v, ii, I}
	**Bolt row below the beam flange** If g > 0.7B_b or T_b < 0.8t_p Use: $\mathrm{Min}\left\{\mathrm{Max}\left\{\left(\frac{ii+ii}{2}\right),ii\right\},i\right\}$ 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}

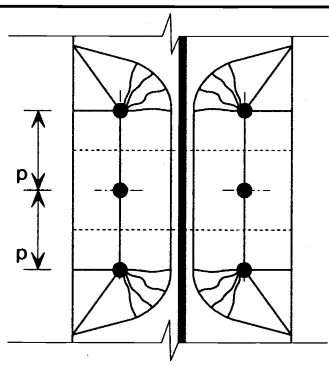
Notes:

- Effective length expressions are given in Table 4.4.
- Any other bolts, above or below are ignored when considering a single bolt row.
- The expressions signified by the pattern numbers determine the 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 a **maximum** from patterns (ii) and (iii), then take the **minimum** of this result and pattern (iI), e.g. if patterns (iI), (ii) and (iii) gave lengths of 300 mm, 200 mm, 100 mm respectively, then the result would be 200 mm.

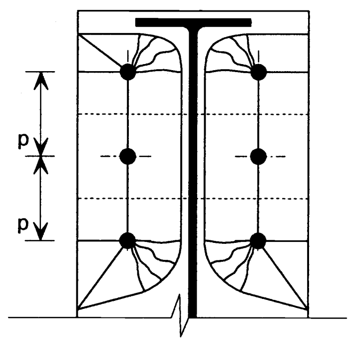
Table 4.6 L_{eff} to be considered for bolt rows acting in combination (expressions to be used from Table 4.4)

	**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: $\mathrm{Min}\left\{e_x,\frac{ii}{2}\right\}+\frac{p}{2}$
 plate thickness t_p	**Bolt row of a group below the beam flange** 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: $\mathrm{Max}\left\{\frac{ii}{2},\left(iii-\frac{ii}{2}\right)\right\}+\frac{p}{2}$		

Typical Examples



Group of three rows in a clear length
L_{eff} = pattern (ii) + $2p$
$\therefore L_{eff}$ for group = 4 m + 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 + $2p$

Notes:
- Effective length expressions for individual rows are given in Table 4.4.
- 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

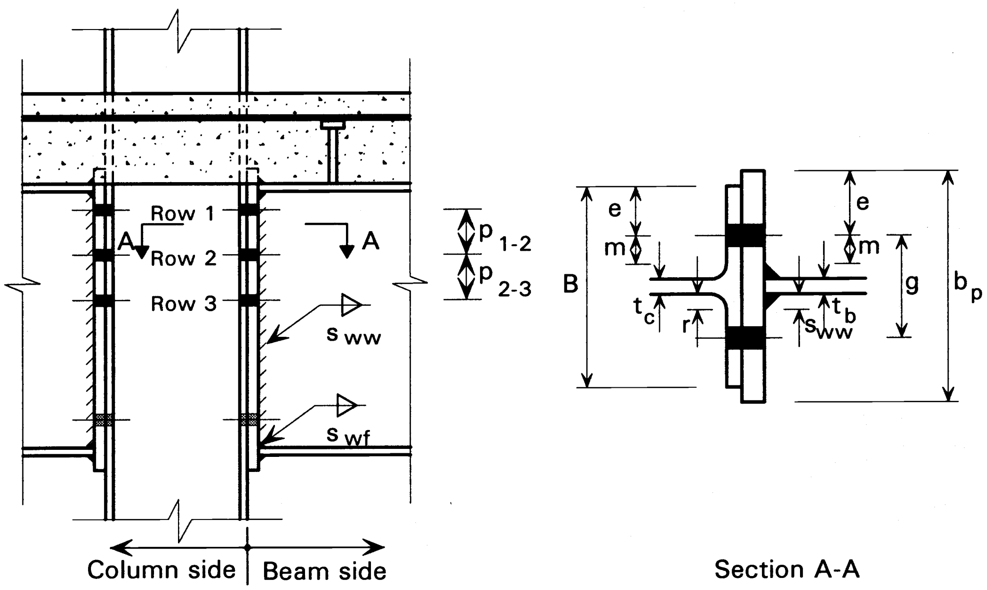


Figure 4.8 Connection geometry

For the end plate:

$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8\, s_{ww} \quad (1.9)$$

$$e = \frac{b_p}{2} - \frac{g}{2} \quad (1.10)$$

For the column flange:

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8\, r \quad (1.11)$$

$$e = \frac{B}{2} - \frac{g}{2} \quad (1.12)$$

The effective edge distance, dimension '*n*' used in the Mode 2 formula, is taken as:

for an end plate, the minimum of

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

for a column flange, the minimum of:

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

where:

g = horizontal distance between bolt centrelines (gauge)

b_p = end plate width

B = column flange width

t_b = beam web thickness

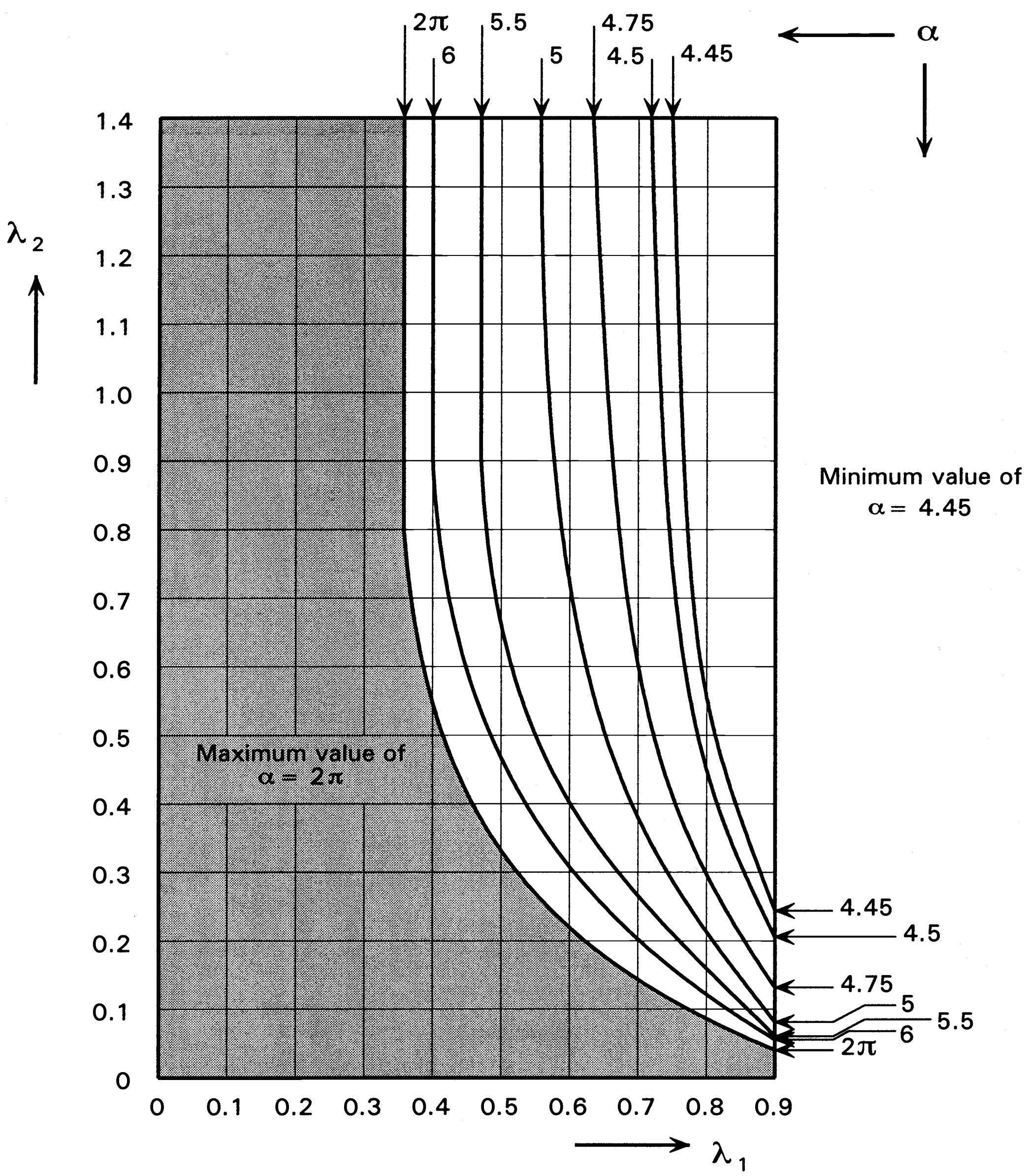
t_c = column web thickness

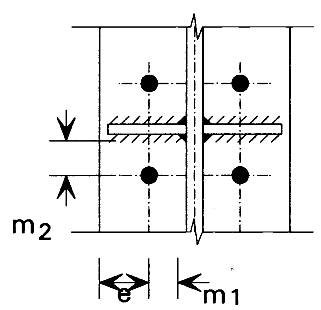
s_{ww} = leg length of fillet weld to beam web

s_{wf} = leg length of fillet weld to beam flange.

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

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 Reference 4

Figure 4.9 Values of α for bolt rows adjacent to a stiffener or beam flange

STEP 1C WEB TENSION IN BEAM OR COLUMN

Note: Web tension failure is **non-ductile** and therefore **must not be allowed** to govern the moment capacity of a **'plastic'** connection.

General

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

$$P_t = L_t \times t_w \times p_y \quad (1.13)$$

where:

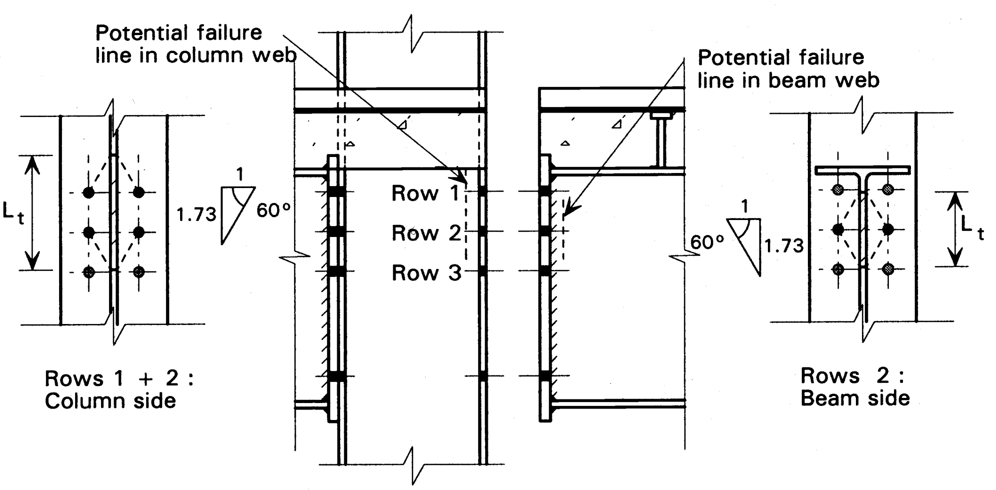
L_t = effective length of web assuming a maximum spread at 60° from the bolts to the centre of the web (Figure 4.10)

t_w = thickness of the 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 within the tensile length, L_t.



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

Figure 4.10 Typical web tension checks

STEP 1D MODIFICATION OF BOLT ROW RESISTANCES

The method given in Steps 1A, 1B and 1C for determining potential resistances in the tension zone is based on a plastic distribution of reinforcement and bolt forces. This distribution is only appropriate when sufficient deformation can take place to plastify the end plate or column flange adjacent to the bolts, and yield the reinforcement.

To ensure sufficient end plate/column flange deformation at working values of rotation, a bolt row must be at least 200 mm above the plastic neutral axis (top of compression zone). The limit of 200 mm is derived from test results. Because the position of the plastic neutral axis is calculated later, in Step 2B, iteration may be required to establish appropriate bolt row resistances. The worked example (Appendix A) demonstrates how the procedure operates in practice.

For bolt rows that are less than 200 mm above the neutral axis (but still in tension), a triangular limit must be imposed to establish the resistance. For example, the resistance of a row 150 mm from the neutral axis would only be three quarters of the full potential resistance (see Figure 4.11).

The detailing requirements given in Section 5 must also be respected to validate the use of the full bolt row resistance. Readers familiar with Reference 4 may be interested to note that satisfying these detailing requirements makes an explicit check of Step 1C in that publication unnecessary.

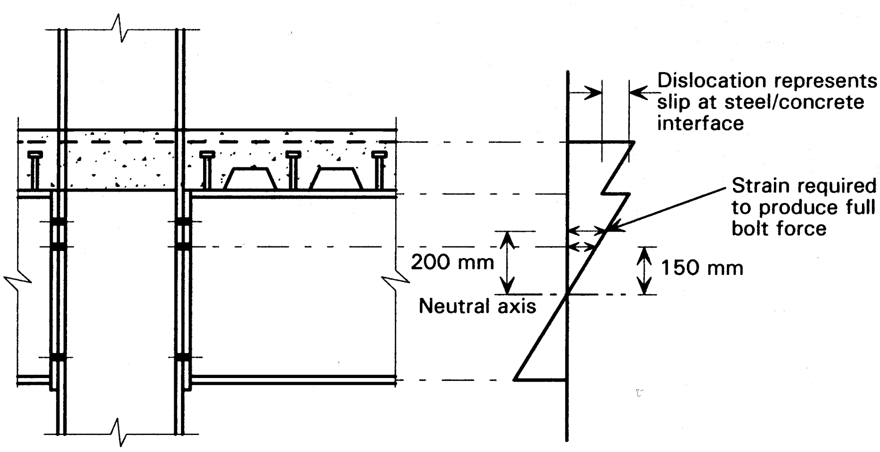


Figure 4.11 Triangular limit to bolt forces

STEP 1 WORKSHEET: TENSION ZONE

Row	Column Side: Step 1B Flange Bending	Column Side: Step 1C Web Tension	Beam Side: Step 1B Plate Bending	Beam Side: Step 1C Web Tension	Step 1D Modify bolt forces	Potential Resistance
	Reinforcement					From Step 1A P_{reinf} = [1]
1	*Resistance of Row 1*					least of boxes 2 to 6 gives
	[2]	[3]	[4]	[5]	[6]	P_{r1} = [7]
2	*Resistance of Row 2 alone*					least of boxes: 8 to 11 and 16 to 20 gives
	[8]	[9]	[10]	[11]		
	Resistance of rows (1 + 2) combined:					
	[12]	[13]	[14]	[15]		
	Deduct box 6:					
	[16]	[17]	[18]	[19]	[20]	P_{r2} = [21]
3	*Resistance of row 3 alone*					least of boxes 22 to 25 and 30 to 33 and 38 to 42 gives
	[22]	[23]	[24]	[25]		
	Resistance of combined rows (2 + 3)					
	[26]	[27]	[28]	[29]		
	Deduct box 20:					
	[30]	[31]	[32]	[33]		
	Resistance of combined rows (1 + 2 + 3):					
	[34]	[35]	[36]	[37]		
	Deduct sum of boxes 6 & 20:					
	[38]	[39]	[40]	[41]	[42]	P_{r3} = [43]

See the worked example using the worksheet in Appendix A

STEP 2A COMPRESSION CHECK - COLUMN
RESISTANCE OF THE COLUMN WEB IN THE COMPRESSION ZONE

The resistance in compression of the column web may be governed by bearing or buckling. Checks for both cases are given below.

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

Note

Column web crushing (bearing) and buckling are both **non-ductile** failure mechanisms, which cannot therefore be allowed to govern the moment capacity of a 'plastic' connection.

Column Web Crushing (Bearing)

The area of web providing resistance to crushing is calculated assuming a force dispersion length as shown in Figure 4.12 (BS 5950: Part 1: Cl 4.5.3).

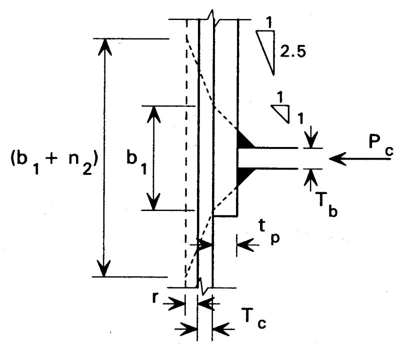


Figure 4.12 Force dispersion for web crushing

$$P_c = (b_1 + n_2) \times t_c \times p_y \qquad (1.14)$$

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

The area of web providing resistance to buckling is calculated assuming a web length as shown in Figure 4.13 (BS 5950: Part 1: Cl 4.5.2.1).

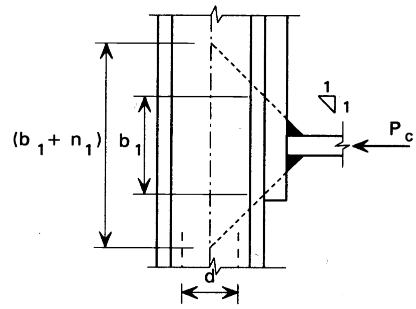


Figure 4.13 Length for web buckling

$$P_c = (b_1 + n_1) \times t_c \times p_c \qquad (1.15)$$

where:

b_1 = stiff bearing length as above
n_1 = length obtained by a 45° dispersion through half the depth of the column, which is equal to the 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, 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)

As a first check, the potential resistance of the flange in compression is taken as:

$$P_c = 1.4 \times p_{yb} \times T_b \times B_b \qquad (1.16)$$

where:

p_{yb} = design strength of the beam

T_b = the beam flange thickness

B_b = the beam flange breadth (with due consideration of any notching)

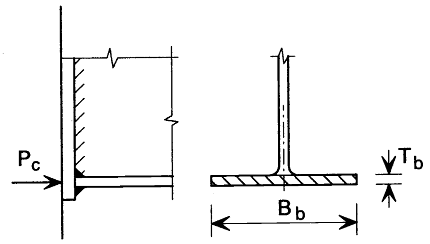


Figure 4.14 Compression in beam flange only

The centre of compression is taken as the mid-depth of the beam compression flange, as shown in Figure 4.14. This accords with the behaviour of connections under test[24].

Allowing the flange-only compression stress to exceed the yield stress by 40% is justified by two localised effects; 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[4]. This value should only be used when flanges are either Class 1 (plastic) or 2 (compact).

When the compression capacity of the flange alone is exceeded, as it often will be due to the presence of significant reinforcement, a ⊥ shaped compression zone should be considered, extending some distance up the web (see Figure 4.15).

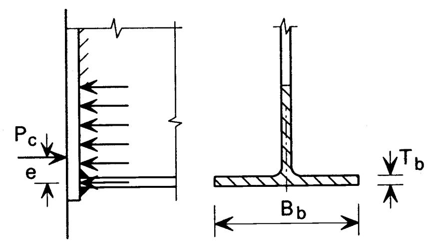


Figure 4.15 Compression in beam flange and portion of web

Note that:

- The stress in this ⊥ section must be limited to $1.2p_y$, since the contribution of the web is now being taken into account explicitly. Flanges should be either Class 1 (plastic) or 2 (compact).
- The centre of compression is redefined as the centroid of the ⊥ section, and the lever arms to the bolts and reinforcement are reduced accordingly.
- An iterative calculation process will become necessary if the height of the neutral axis is such that bolt resistances need modifying (Step 1D). The worked example (Appendix A) demonstrates how the procedure operates in practice.

Because compression in the beam extends into the web, and a limiting stress of $1.2p_y$ assumes plastification to take place, the web should be Class 1 (plastic) or 2 (compact). Because connection detailing rules limit the depth of compression to the lower half of the beam, all practical rolled sections effectively satisfy this restriction. If fabricated sections with a Class 3 (semi-compact) web are used, they may be treated as Class 2 by ignoring part of the web. The effective section should be defined by considering a depth of web equal to $19t_c\epsilon$ adjacent to both the compression flange and the neutral axis (where t_c is the web thickness and $\epsilon = \sqrt{(p_y/275)}$).

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 is:

$$P_v = 0.6 \times p_{yc} \times t_c \times D_c \qquad (1.17)$$

where

p_{yc} = design strength of the column

t_c = column web thickness

D_c = column section depth

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

For a two-sided connection with balanced moments, the shear is zero, but in the case of a connection with unbalanced moments the shear is the difference between the two opposing forces.

The resistance of stiffened columns can be determined by reference to Step 6D.

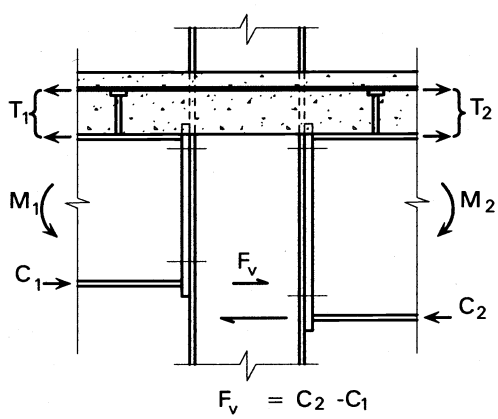


Figure 4.16 Web panel subject to shear force

STEP 4 CALCULATION OF MOMENT CAPACITY

Force Distribution

The reinforcement and bolt row resistances calculated in Step 1 can only be fully realised if sufficient compression zone resistance is available (Step 2). They are therefore 'potential' resistances, and must be reduced if necessary to ensure equilibrium. Figure 4.17 shows the transition from potential resistances (P) into actual forces (F).

To maintain equilibrium, the sum of the tensile forces must satisfy the following equation:

$$F_{reinf} + \sum F_{ri} + N \quad = F_c \qquad (1.18)$$

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

and F_c is the smallest of the following:

$$P_{reinf} + \sum P_{ri} + N \qquad (1.19)$$

or P_c *(column web crushing (bearing))*. Note that this is an unacceptable failure mechanism for a 'plastic' connection, and must not govern moment capacity in such cases.

or P_c *(column web buckling)*. Note that this is an unacceptable failure mechanism for a 'plastic' connection, and must not govern moment capacity in such cases.

or P_c *(beam crushing (bearing))*

If the first of these conditions limits the actual tensile forces it means that the full 'potential' resistances can be mobilised. If one of the latter three conditions governs, it means that the full 'potential' resistances cannot be achieved because of compression limitations.

If there is an excess potential resistance in the reinforcement or bolts in tension, then the forces should be reduced. Reduction should start with the bottom row of bolts and work up progressively until equilibrium is achieved between the tension and compression forces.

For the reinforcement: $F_{reinf} \leq P_{reinf}$

For each bolt row: $F_{ri} \leq P_{ri}$

where:

P_{reinf} = potential resistance of reinforcement

P_{ri} = potential resistance of bolt row i

F_{reinf} = final force in reinforcement

F_{ri} = final force in bolt row i

Column web panel shear requirements must also be satisfied (see Step 3).

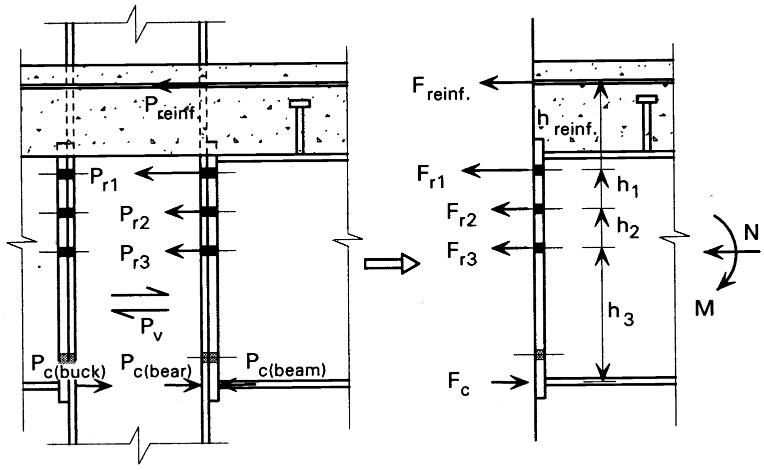


Figure 4.17 Translation of potential resistances into allowable forces

STEP 4 CALCULATION OF MOMENT CAPACITY (Continued)

Moment Capacity

Assuming that there is no significant axial load in the beam, the basic moment capacity of the connection is given by:

$$M_j = F_{reinf} \times h_{reinf} + \sum(F_{ri} \times h_i) \qquad (1.20)$$

where:

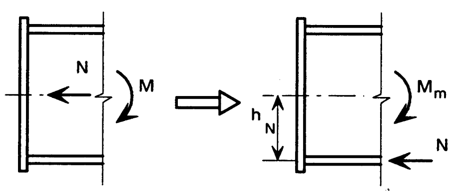
h_{reinf} = distance from the centre of compression to the reinforcement

h_i = distance from the centre of compression to bolt row I.

Moment Capacity Modified by Axial Load

The presence of significant axial load must be allowed for when checking equilibrium between the tension and compression forces (Equation 1.18). For simplicity in the connection design, it is assumed that the axial force acts at the centre of compression in the beam. However, this shift in the line of action of the force, from the (composite) beam neutral axis, means that the coincident moment that is considered to be applied to the connection should be reduced. The total applied moment (M) is therefore considered as comprising a reduced moment (M_m) plus an eccentric axial force ($N \times h_N$).

The presence of axial load is therefore allowed for conservatively in the moment capacity calculation procedure by considering the axial load in the check on compression capacity (Equation 1.18), and increasing the connection moment capacity to compensate for this conservatism:



$$M_{jm} = M_j + (N \times h_N) \qquad (1.21)$$

where:

M_j = basic connection moment capacity

N = axial force

h_N = distance of axial force from centre of compression

STEP 5 DESIGN FOR VERTICAL SHEAR FORCES

Comprehensive capacity checks for steel end plate connections subjected to vertical shear are given in *Joints in simple construction, Volume 1*[1].

However, for full depth fully welded end plates 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 the full shear strength for bolt rows which are ignored when calculating moment capacity. The slab and reinforcement are assumed to make no contribution to the shear capacity.

Major axis connections

It is required that:

$$V \leq (n_s \times P_{ss}) + (n_t \times P_{ts}) \qquad (1.22)$$

where:

V = design shear force

n_s = number of shear bolts

n_t = number of tension bolts

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 end plate, or

$d\, T_c\, p_b$ 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 end plate, 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

t_p = end plate thickness

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)

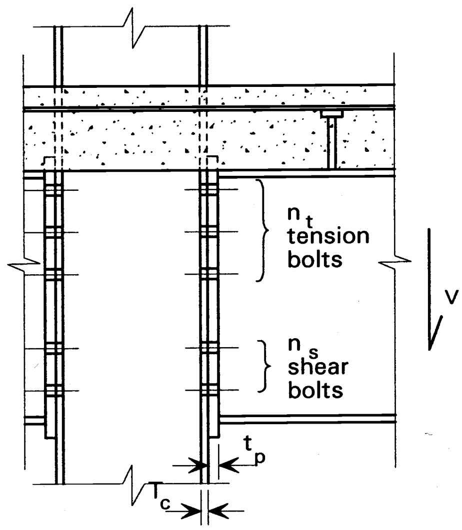


Figure 4.18 Tension and shear bolts

Table 4.7 Shear capacities of single 8.8 bolts

Bolt size	Bolts in shear only kN	Bolts in shear & tension 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 4.8 for bearing capacities.

Minor axis connections

The following rules only apply to double sided minor axis composite connections. If required, information concerning single sided minor axis connections may be found in Reference 3. Checks (i) to (iv) must all be satisfied.

(i) For column web shear:

$$V_1 + V_2 \leq 2P_v \qquad (1.23)$$

V_1 = shear applied from beam 1

STEP 5 DESIGN FOR VERTICAL SHEAR FORCES (continued)

V_2 = shear applied from beam 2

P_v = local shear capacity of column web
= smaller of $0.6p_yA_v$ and $0.5U_sA_{vnet}$

$A_v = [p + (n_t + n_{s2} - 2)p/2 + e_t]t_c$ *(1.24)*

p = bolt pitch

n_{s2} = number of shear bolts on side 2

$e_t \leq 5d$

d = bolt diameter

t_c = column web thickness

$A_{vnet} = A_v - (n_{s2} + n_t)D_ht_c/2$ *(1.25)*

D_h = hole diameter

U_s = ultimate tensile strength of column web

(ii) For bolt shear:

$V_1 \leq (n_{s1} \times p_sA_s) + (n_t \times 0.4p_sA_s)$ *(1.26)*

and

$V_2 \leq (n_{s2} \times p_sA_s) + (n_t \times 0.4p_sA_s)$ *(1.27)*

where:

p_s = strength of the bolt in single shear

n_{s1} = number of shear bolts on side 1 (see Figure 4.19)

(iii) For end plate bearing:

$V_1 \leq (n_{s1} + n_t)\, dt_{p1}p_b$ *(1.28)*

and

$V_2 \leq (n_{s2} + n_t)\, dt_{p2}p_b$ *(1.29)*

where:

t_{p1} = end plate thickness on side 1

t_{p2} = end plate thickness on side 2

p_b is the minimum of p_{bb} (bolt) or p_{bs} (plate)

(iv) For web bearing:

$V_1/(n_{s1} + n_t) + V_2/(n_{s2} + n_t) \leq dt_cp_b$ *(1.30)*

where:

p_b is the minimum of p_{bb} (bolt) or p_{bs} (web)

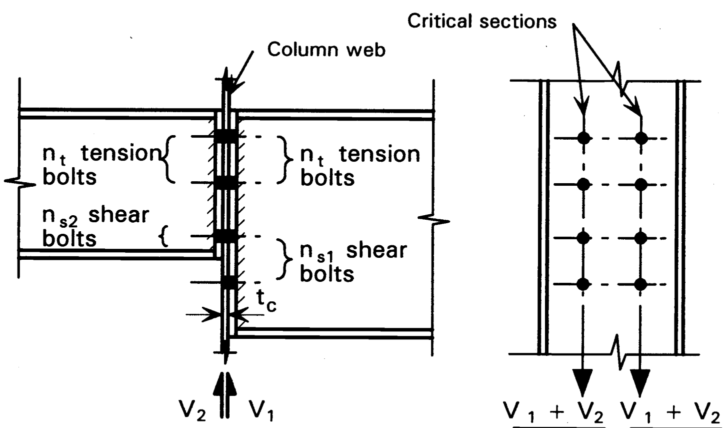


Figure 4.19 **Shear in minor axis beam-to-column connection** (slab not shown for clarity)

STEP 5 DESIGN FOR VERTICAL SHEAR FORCES (continued)

Table 4.8 Capacities for ordinary bolts to BS 3692 and BS 4190*

Capacities in kN for bolts in 2 mm clearance holes ≤ 24 mm diameter
3 mm clearance holes > 24 mm diameter

8.8 bolts passing through design grade S275 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											
		BS5950: 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
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

8.8 bolts passing through design grade S355 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											
		BS5950: 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
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	

* *Although still commonly used, these standards have been replaced by:*
Bolts : BS EN 24014 and 24016
Nuts : BS EN 24032, 24033 and 24034
Screws : BS EN 24017 and 24018

Values given in the table are applicable to both old and new standards.

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 4.23 is the lower value from Equations 1.31 and 1.32 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 1.33. This is usually the formula which governs.

These rules are taken from BS 5950: Part 1: 1990, and are likely to be amended in subsequent editions of the British Standard. Appropriate modifications should be made to these procedures as necessary.

Stiffeners are usually designed in S275 steel. It is recommended that they be at least as thick as the beam flange, and specified in thicknesses of 15, 20 or 25 mm only.

Note that the presence of stiffeners may interfere with erection of orthogonal beams, whose end plates may themselves increase the buckling strength of the column web.

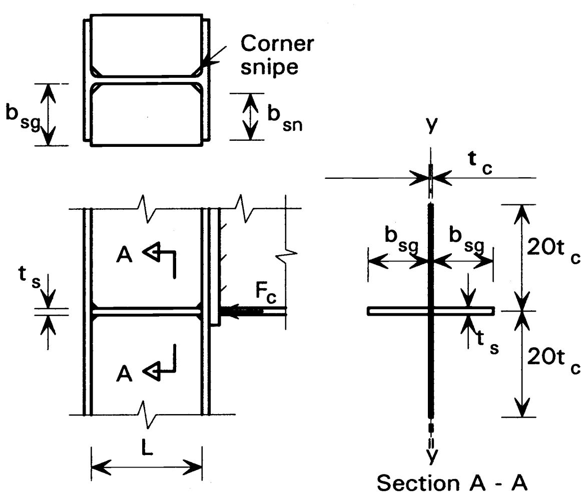


Figure 4.20 Stiffener bearing and buckling

Effective Outstand of Compression Stiffeners

The outstand of S275 compression stiffeners b_{sg} should not exceed $19t_s$ (see Figure 4.20)

where:

b_{sg} = stiffener outstand (see Figure 4.20)
t_s = thickness of stiffener

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

Stiffener/Column Web Crushing and Buckling

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

$$P_{c\ buckling} = (A_w + A_{sg}) \times p_c \qquad (1.31)$$

$$P_{c\ crushing} = [A_{sn} \times p_y] + [(b_1 + n_2) \times t_c \times p_y] \qquad (1.32)$$

and:

$$P_{c\ bearing} = \frac{A_{sn} \times p_{ys}}{0.8} \qquad (1.33)$$

where:

A_w = allowable area of column web for buckling (see Section A-A in Figure 4.20), $\leq 40\ t_c \times t_c$

A_{sg} = gross area of stiffeners

$= 2 \times b_{sg} \times t_s \qquad (b_{sg} \leq 13t_s) \qquad (1.34)$

A_{sn} = net area of stiffeners in contact with column flange

$= 2 \times b_{sn} \times t_s \qquad (1.35)$

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

STEP 6A DESIGN OF COLUMN COMPRESSION STIFFENERS (continued)

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

r_y = radius of gyration of effective area (as shown in Section A-A, Figure 4.20)

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 2A)

Note: 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: Part 1 Clause 4.5.1.5.*

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: Part 1 Clauses 4.5.9 and 4.5.11 as follows:

Welds to Flanges

When, as is normally the case, the stiffener is fabricated to achieve a bearing fit to the inside of the column flange, the weld to the flange need only be nominal, say a 6 mm fillet weld. In other cases, the welds should be designed as full strength.

Welds to Web

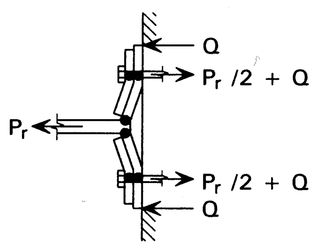
For double sided connections, the web welds must be designed to carry the larger of the beam flange forces, assuming that the forces act in opposite directions.

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 Equations 1.36, 1.37 and 1.38.

Mode 1 Complete Flange Yielding

$$P_r = \frac{4M_p + 2M_{bp}}{m} \qquad (1.36)$$



Mode 2 Bolt failure with flange yielding

$$P_r = \frac{2M_p + n\left(\Sigma P_t'\right)}{m+n} \qquad (1.37)$$

Mode 3 Bolt failure

$$P_r = \Sigma P_t' \qquad (1.38)$$

where:

$$M_{bp} = \frac{L_{eff} \times t_{bp}^2 \times p_y}{4} \qquad (1.39)$$

t_{bp} = thickness of the backing plate

p_y = design strength of the backing plate

other variables are defined in Step 1B.

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 sufficient so that the plate extends not less than 2*d* beyond the bolts at its extremities (*d* is the bolt diameter).

This type of strengthening is only useful for smaller section columns were 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. Neither Mode 2 nor Mode 3 is affected.

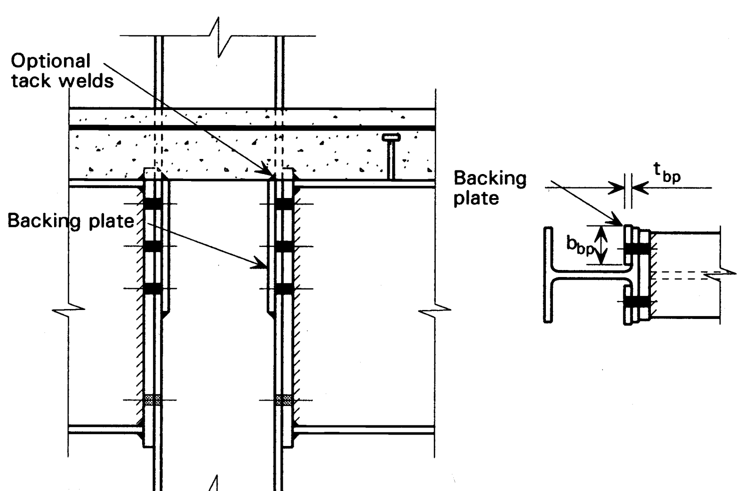


Figure 4.21 Column flange backing plates

STEP 6C DESIGN OF TENSION STIFFENERS

General

Tension stiffeners, as shown in Figure 4.22, are generally used to supplement the tension capacity of the column web and/or the capacity of the column flange in bending. Although it is recommended that the stiffeners be full depth, design rules for partial depth stiffeners may be found in Reference 4.

From a practical point of view, the presence of column web stiffeners may interfere with the erection of orthogonal beams.

Stiffener Net Area

The net area of the stiffeners, A_{sn} must satisfy the requirements of Equations 1.40 and 1.41 for web tension and flange bending respectively.

Web Tension

The stiffener, in combination with the column web, is designed to carry the tensile load from the bolts immediately above and below it.

Basic requirement:

$$A_{sn} \geq \frac{(F_{ri} + F_{rj})}{p_y} - (L_t \times t_c) \qquad (1.40)$$

where:

A_{sn} = net area of both stiffeners

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

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 maximum spread of load at 60° from the bolts - the spread may be limited by the presence of adjacent bolts (see Figure 4.23)

t_c = web thickness

The gross width of stiffener, b_{sg} should generally be proportioned so that the stiffener extends at least 75% across the available flange width, $(B_c - t_c)/2$

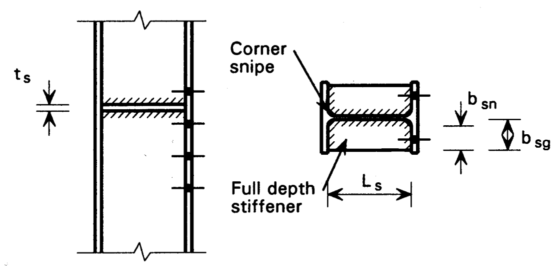


Figure 4.22 Tension stiffeners

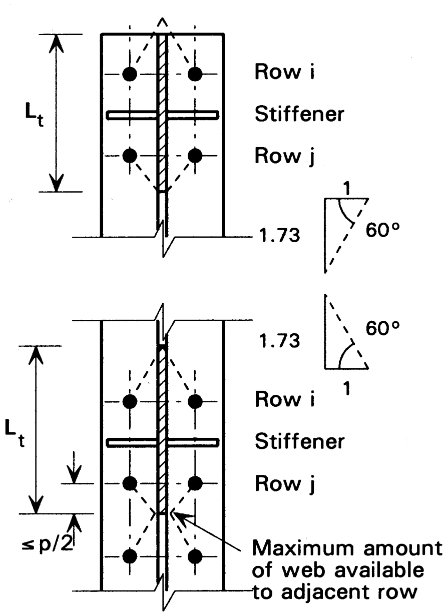


Figure 4.23 Effective web lengths

Flange Bending

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

STEP 6C DESIGN OF TENSION STIFFENERS (continued)

Basic requirement:

$$A_{sn} \geq \left[\frac{F_{ri}}{(m_1 + m_{2L})} + \frac{F_{rj}}{(m_1 + m_{2U})} \right] \qquad (1.41)$$

where:

A_{sn} = net area of both stiffeners

= $2\ (b_{sn} \times t_s)$ *(1.42)*

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

m_1 m_{2L} m_{2U} F_{ri} and F_{rj} are defined in Figure 4.24

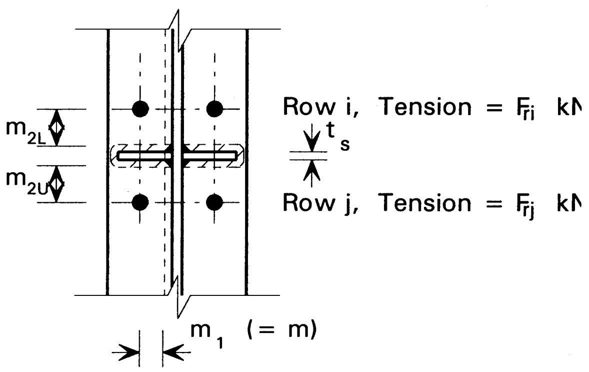


Figure 4.24 Geometry for force distribution to stiffeners

Weld Design

Full strength welds should be provided to the flanges and web.

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 (according 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 satisfy the following criteria:

- **Thickness**, t_s not less than the column web thickness.
- The same **material strength** as the column.
- **Welds** all round should be, as a minimum, fillet welds of leg length equal to the plate thickness. 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 4.25). Plug welds are required if b_s exceeds $37t_s$ (for S275 steel) or $33t_s$ (S355).
- **Breadth**, b_s that satisfies:
  $b_s \geq d - 2t_s$ *(1.43)*
  ($b_s = d$ for a "fill in" weld)
- **Length,**
  $L_s \geq g + D_b + D_c/2 + D_r + e$ *(1.44)*

  where:

g	=	horizontal spacing of bolts (gauge)
D_b	=	depth of beam
D_c	=	depth of column
D_r	=	depth to reinforcement
e	=	projection of end plate beyond beam (25 mm for standard details)

Column Web Tension

For the purpose of the column web tension calculation (Step 1C), the effective web thickness, t_{eff}, should be taken as:

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

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

t_c is the column web thickness.

Column Web Crushing and Buckling

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

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

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

t_c is the column web thickness.

Column Panel Shear

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

$$P_v = 0.6 \times p_y \times A_v \qquad (1.45)$$

where:

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

There is no further increase in the shear area if an SWP is added on the other side of the web.

STEP 6D DESIGN OF SUPPLEMENTARY WEB PLATES (continued)

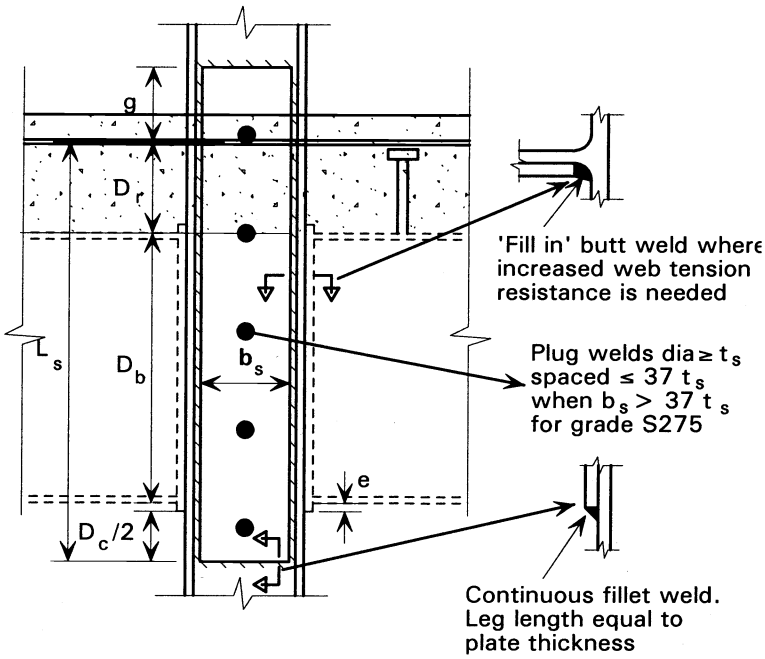


Figure 4.25 Dimensions and welds

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,
$= B \times T \times p_y$ *(1.46)*

(b) The total tension force in the top two bolt rows (see Figure 4.26)
$= F_{r1} + F_{r2}$ *(1.47)*

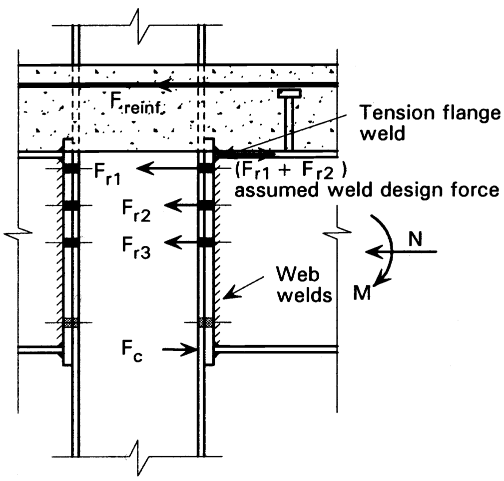


Figure 4.26 Forces in welds

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 thicknesses 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 4.27, note that:

- The weld throat required should be calculated based on the strengths given in BS 5950: Part 1 Table 36, (i.e. 215 N/mm^2 for S275 and 255 N/mm^2 for S355).
- The shear and tension stresses 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 3 mm 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 4.27.

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

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

Although the approach given above may appear conservative, 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 simply that due to the adjacent bolts. 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.

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 8 mm fillet welds will suffice. For some of the lighter beams (with flange thicknesses of 12 mm or less) 6 mm fillet welds may be appropriate. This 'bearing' assumption will be the usual case for most plain beams. Guidance on the necessary tolerances for a bearing fit can be found in the NSSS[9].

STEP 7 DESIGN OF WELDS (continued)

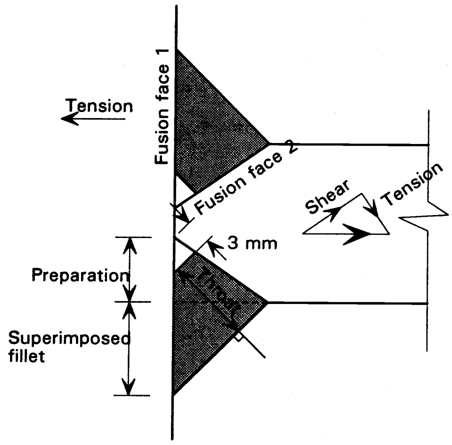


Figure 4.27 A partial penetration butt weld with superimposed fillets

If a bearing fit cannot be assumed, the weld must be designed to carry the full compressive force, F_c.

Web Welds

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

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

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

1. ***Tension Zone*:**
Full strength welds should be used. These will be generally fillet welds with the sum of the throat thicknesses not less than the web thickness, t_b.

The welds should extend below the bottom bolt row resisting tension by a distance of 1.73g/2 (see Figure 4.28). Dimension g is the gauge of the bolts.

2. ***Shear Zone*:**
The capacity of the beam web welds for vertical shear should be taken as:

$$P_{sw} = 2 \times a \times p_w \times L_{ws} \qquad (1.48)$$

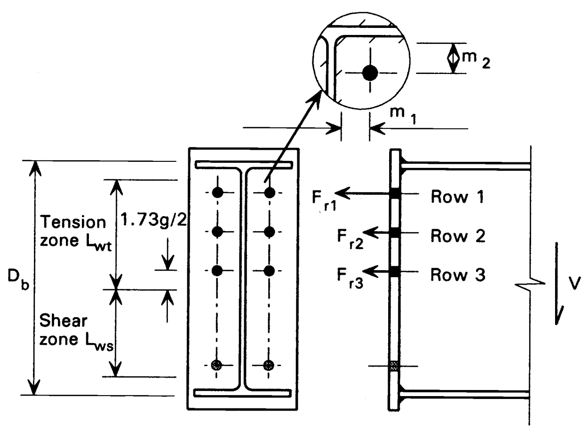
where:

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

$$= D_b - 2\,(T_b + r_b) - L_{wt} \qquad (1.49)$$



Note: The tension zone welds are assumed to start at the bottom of the root radius.

Figure 4.28 Force distribution in welds

4.3 BEAM-TO-BEAM CONNECTIONS

Beam-to-beam connections are usually detailed such that notching of the supported beams allows their top flanges to be made level with that of the support (see Figure 4.29). The supported beams must then adopt a partial depth end plate. The tension bolt forces that can be generated with such a detail are low, and, for simplicity, can be ignored when calculating the composite connection moment capacity without excessive simplicity. The connections should be treated as pinned at the construction stage. A range of connections of this type has been tested in order to validate the design procedures presented in this publication. Alternative detailing should be considered if there is a need to generate significant tension forces in the steelwork.

As in connections to column webs, it is important that there is a direct load path to transfer compressive forces between the bottom flanges of opposing beams. It is therefore recommended that the two supported beams should be of the same serial size. Because the torsional stiffness of the supporting beam is relatively low compared with the flexural stiffness of the supported beams, the latter are effectively treated as a semi-continuous beam on a knife edge support.

The following design steps, derived from those given in Section 4.2 and maintaining the same numbering system, should be used to calculate the moment capacity of a beam-to-beam connection.

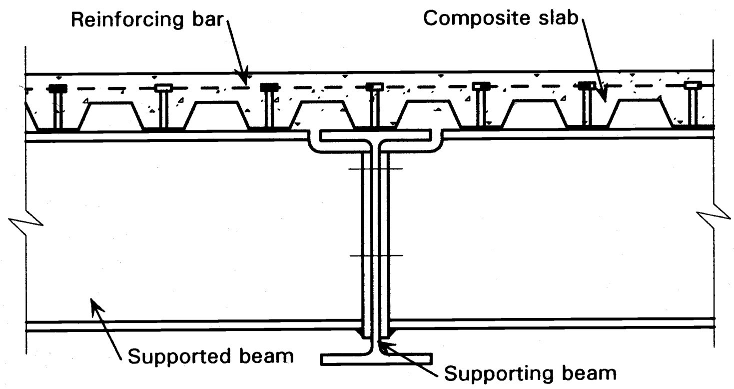


Figure 4.29 A typical beam-to-beam connection

STEP 1A REINFORCEMENT YIELDING

Potential resistance

The potential resistance of the reinforcement in tension is given by:

$$P_{reinf} = \frac{f_y A_{reinf}}{\gamma_m} \qquad (2.1)$$

f_y = design yield strength of reinforcement

A_{reinf} = area of reinforcement within the allowable width of slab (see Figure 5.2).

γ_m = partial safety factor for reinforcement (taken as 1.05)

Detailing rules for the reinforcement are given in Section 5.

Minimum area of reinforcement

In general, the rotation capacity of a connection increases as the area of reinforcement increases. A minimum area is therefore needed to ensure that 'compact' connections can undergo sufficient rotation to strain the reinforcement to yield (as is assumed in the moment capacity model). Minimum areas of reinforcement are given in Table 4.1 as a function of:

- beam size
- beam steel grade
- reinforcement properties
- connection type - 'compact' or 'plastic'.

The minimum reinforcement limits in Table 4.1 are more onerous for 'plastic' connections, because in addition to the need for yielding of the reinforcement, the connection must have sufficient rotation capacity to behave as a plastic hinge.

For a given beam size and steel grade, the minimum values in Table 4.1, marked '5%', should generally be used. These values are appropriate for high yield bars complying with current British Standard BS 4449 grade 460B[20].

This grade of reinforcement has a mandatory requirement of 14% minimum elongation at fracture, and a non-mandatory requirement of 5% minimum elongation at maximum force. Grade B500B bars complying with BS EN 10080[21] are required to have similar properties; they must be able to achieve 5% total elongation at maximum force. Elongation at fracture and total elongation at maximum force are defined in Figure 4.5.

Minimum reinforcement limits are also given in Table 4.1 for connections that use reinforcement which is capable of achieving 10% minimum elongation at maximum force. The increased reinforcement ductility offers considerable advantages in some cases, because it permits the use of less reinforcement.

It is essential that when a design is based on the use of 10% elongation bars this is made clear in the project specification. This can be done by giving the bars an 'X' designation, rather than the 'T' generally used for high tensile bars[22]. The 'X' informs the contractor that the bars need specific, non-standard properties. It is recommended that, if possible, the reinforcement supplier uses coloured labels to clearly distinguish the high elongation 'X' bars on site. In case of doubt concerning the elongation capacity of bars, approximately half the UK manufacturers provide reinforcement suppliers with appropriate test information. It should therefore be relatively easy for the contractor to confirm suitability with his reinforcement supplier.

Bars that are currently produced in the UK using a hot forming process may be assumed to be appropriate for use with the '10%' limits. All 20 mm diameter bars produced by major manufacturers in the UK currently are hot formed, as are, often, 16 mm bars.

Maximum area of reinforcement

The reinforcement area in a beam-to-beam connection must be limited to a maximum value in order to keep the compression zone in the lower half of the steel beam. This restriction ensures that the reinforcement strains sufficiently to produce yielding.

STEP 2B COMPRESSION CHECK - SUPPORTED BEAM

RESISTANCE OF THE BEAM FLANGE AND WEB IN THE COMPRESSION FLANGE

Beam Flange Crushing (Bearing)

As a first check, the potential resistance of the flange in compression is taken as:

$$P_c = 1.4 \times p_{yb} \times T_b \times B_b \qquad (2.2)$$

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 the mid-depth of the beam compression flange, as shown in Figure 4.30. This accords with the behaviour of connections under test[24].

Allowing the flange-only compression stress to exceed the yield stress by 40% is justified by two localised effects; 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[4]. This value should only be used when flanges are either Class 1 (plastic) or 2 (compact).

When the compression capacity of the flange alone is exceeded, as it often will be due to the presence of significant reinforcement, a ⊥ shaped compression zone should be considered, extending some distance up the web (see Figure 4.31).

Note that:

- The stress in this ⊥ section must be limited to $1.2p_y$, since the contribution of the web is now being taken into account explicitly. Flanges should be either Class 1 (plastic) or 2 (compact).
- The centre of compression is redefined as the centroid of the ⊥ section, and the lever arms to the bolts and reinforcement are reduced accordingly.

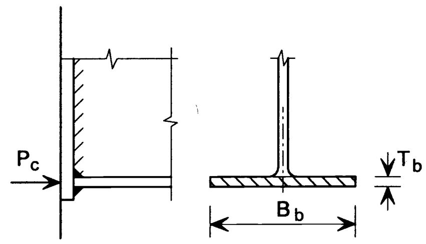


Figure 4.30 Compression in beam flange only

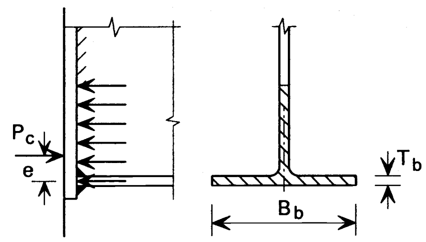


Figure 4.31 Compression in beam flange and portion of web

Because compression in the beam extends into the web, and a limiting stress of $1.2p_y$ assumes plastification to take place, the web should be Class 1 (plastic) or 2 (compact). Because connection detailing rules limit the depth of compression to the lower half of the beam, all practical rolled sections effectively satisfy this restriction. If fabricated sections with a Class 3 (semi-compact) web are used, they may be treated as Class 2 by ignoring part of the web. The effective section should be defined by considering a depth of web equal to $19t_c\epsilon$ adjacent to both the compression flange and the neutral axis (where t_c is the web thickness and $\epsilon = \sqrt{(p_y/275)}$).

STEP 4 CALCULATION OF MOMENT CAPACITY

Force Distribution

The reinforcement resistance calculated in Step 1 can only be fully realised if sufficient compression zone resistance is available (Step 2). It is therefore a 'potential' resistance, and must be reduced if necessary to ensure equilibrium. Figure 4.32 shows the potential resistances (P) translated into actual forces (F).

Equilibrium is satisfied by:

$$F_{reinf} + N = F_c \qquad (2.3)$$

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

and F_c is the smaller of the following: $P_{reinf} + N$

or P_c *(beam crushing)*

For the reinforcement: $F_{reinf} \leq P_{reinf}$

where:

F_{reinf} = final force in reinforcement

If there is an excess potential resistance in the reinforcement then the forces should be reduced until equilibrium is achieved between the tension and compression forces.

Moment Capacity

Assuming that there is no significant axial load in the beam, the basic moment capacity of the connection is given by:

$$M_j = F_{reinf} \times h_{reinf} \qquad (2.4)$$

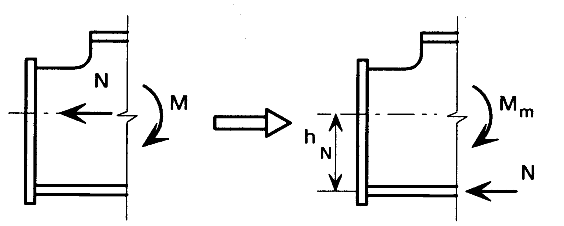
where:

h_{reinf} = distance from the centre of compression to the reinforcement

Moment Capacity Modified by Axial Load

The presence of significant axial load must be allowed for when checking equilibrium between the tension and compression forces (Equation 2.3). For simplicity in the connection design, it is assumed that the axial force acts at the centre of compression in the beam. However, this shift in the line of action of the force, from the (composite) beam neutral axis, means that the coincident moment that is considered to be applied to the connection should be reduced. The total applied moment (M) is therefore considered as comprising a reduced moment (M_m) plus an eccentric axial force ($N \times h_N$).

The presence of axial load is therefore allowed for conservatively in the moment capacity calculation procedure by considering the axial load in the check on compression capacity (Equation 2.3), and increasing the connection moment capacity to compensate for this conservatism:



$$M_{jm} = M_j + (N \times h_N) \qquad (2.5)$$

where:

M_j = basic connection moment capacity

N = axial force

h_N = distance of axial force from centre of compression

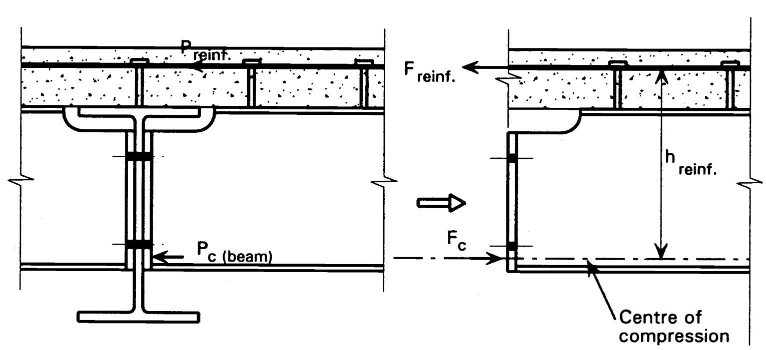


Figure 4.32 Translation of potential resistances into allowable forces

5 CONNECTION DETAILING

5.1 BEAM-TO-COLUMN CONNECTIONS

Appropriate connection detailing for **'compact'** connections is necessary in order to:

- prevent premature failure of the tension bolts or reinforcement
- ensure sufficient deformation takes place to generate the tension bolt and reinforcement forces assumed in the design
- prevent concrete crushing against the column under unbalanced loading.

In addition, for **'plastic'** connections, more onerous detailing rules are necessary in order to:

- ensure that the connections have sufficient rotation capacity to form a plastic hinge.

The detailing rules that follow apply to both 'compact' and 'plastic' connections. They should be used in conjunction with minimum reinforcement area requirements given in Table 4.1, which do however differ for the two types of connection.

Reinforcement and shear connection

Conventional reinforcement detailing according to BS 8110[11] should be adopted. Bar diameter should not be less than 16 mm, since smaller diameter bars are generally less ductile. Effective anchorage of the reinforcement is achieved by curtailing the bars in the compression zone of the slab. This zone normally starts at about 0.2 times the beam span on either side of the support, and sufficient anchorage length should be provided beyond this point, as in conventional reinforced concrete practice[11] (for example 40 times the bar diameter for a 'Type 2 deformed' bar in concrete with a cube strength of 30 N/mm^2). Although reinforcing mesh may also be present in the slab to control cracking, its contribution to moment capacity should be ignored.

The following detailing rules are shown schematically in Figure 5.1. Limitations on the positions of reinforcing bars ensure that they can work effectively as components in a truss to resist unbalanced loads (see Section 4.2 Step 1A).

1. Longitudinal reinforcing bars should be uniformly spaced either side of the column (see Figure 5.1), with the nearest bars approximately 20 mm from the column edge, to achieve adequate cover. The furthest bars to be included in the effective area should not be more than approximately $2b_c$ from the column centreline (dimension e_L).
NOTE: e_L is a function of the column width, rather than the beam span.

2. Transverse reinforcing bars (which are necessary to resist forces in the concrete 'behind' the column when unbalanced loading is applied) should not extend more than the required anchorage length[11] beyond points $2b_c$ either side of the column centre line. This should ensure that the orthogonal connection behaviour is not significantly influenced by these bars.

3. Transverse reinforcing bars should be uniformly spaced either side of the column (see Figure 5.1), with the nearest bars approximately 20 mm from the column edge. The furthest bars included in the effective area should not be more than approximately $3b_c$ from the column face (dimension e_T).

4. Longitudinal reinforcing bars should be placed approximately 20 mm above the top of the decking, to ensure adequate concrete cover. Whether the transverse bars are placed above or below the longitudinal bars will depend on the orientation of the decking and the depth of slab above the decking; all the bars must be positioned so that they have adequate cover to the decking and the top of slab[11].
Keeping the longitudinal bars close to the top of the decking minimises the strain they must undergo to achieve a given rotation.

5. The first shear connector should be at least 100 mm from the face of the column.
This limitation ensures that reinforcing bars are strained over a substantial length, so that sufficient rotation can take place.

Steelwork

6. The end plate thickness should be not more than 60% of the bolt diameter; 12 mm for M20 bolts and 15 mm for M24 bolts. End plates should be made from S275 steel.

7. Horizontal spacing of the bolts (gauge) should be not less than 90 mm.

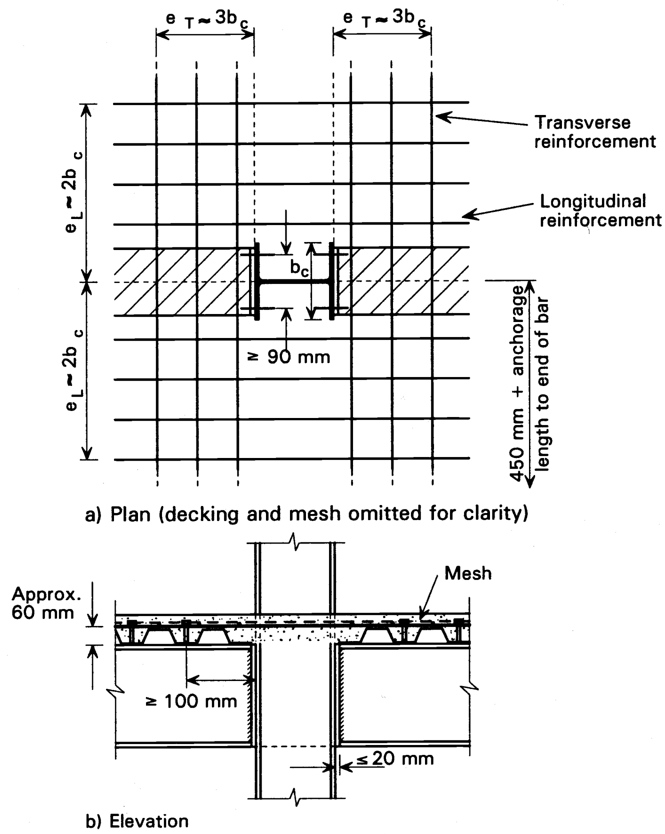


Figure 5.1 Geometrical detailing rules - beam-to-column connections

These steelwork restrictions ensure that, as the connection rotates, end plate deformation is the 'weak link'. Steelwork details complying with these rules have been shown in tests to possess sufficient rotation capacity to satisfy both 'compact' and 'plastic' connection requirements[4]. Alternatively, steelwork details which fail in Mode 1 may be used (see Section 4.2 Step 1B).

5.2 BEAM-TO-BEAM CONNECTIONS

For beam-to-beam details, the following detailing rules should be respected for both 'compact' and 'plastic' connections. These are required for the same reasons as specified in Section 5.1, except that concrete crushing against the column is clearly not relevant.

Reinforcement and shear connection

Conventional reinforcement detailing according to BS 8110[11] should be adopted. Bar diameter should not be less than 16 mm, since smaller diameter bars are generally less ductile. Effective anchorage of the reinforcement is achieved by curtailing the bars in the compression zone of the slab. This zone normally starts at about 0.2 times the beam span on either side of the support, and sufficient anchorage length should be provided beyond this point, as in conventional reinforced concrete practice[11]. Although reinforcing mesh may also be present in the slab to control cracking, its contribution to moment capacity should be ignored.

The following detailing rules are shown schematically in Figure 5.2.

1. Longitudinal reinforcing bars must be located within the effective width of the concrete slab for the beam in hogging, as defined in BS 5950: Part 3: Section 3.1 Clause 4.6.
   Note: Effective width is not the same as for beam-to-column connections.

2. Reinforcing bars should be placed approximately 20 mm above the top of the decking, to maintain adequate cover to the decking and the top of slab[11].

3. The first shear connectors on the supported beams should be at least 200 mm from the centre line of the support.

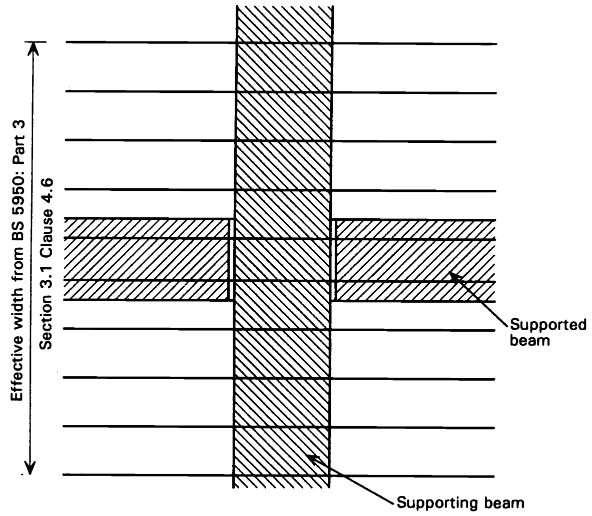


Figure 5.2 Geometrical detailing rules - beam-to-beam connections

Steelwork detailing

4. The end plate thickness should be not more than 60% of the bolt diameter; 12 mm for M20 bolts and 15 mm for M24 bolts. End plates should be made from S275 steel.

5. Horizontal spacing of the bolts (gauge) should be not less than 90 mm.

REFERENCES

1 THE STEEL CONSTRUCTION INSTITUTE and THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION LTD
Joints in simple construction - Volume 1: design methods
SCI, BCSA, 1993

2 THE STEEL CONSTRUCTION INSTITUTE and THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION LTD
Joints in simple construction - Volume 2: practical applications
SCI, BCSA, 1992

3 THE STEEL CONSTRUCTION INSTITUTE and THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION LTD
Joints in steel construction: simple connections
SCI, BCSA, 1998

4 THE STEEL CONSTRUCTION INSTITUTE and THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION LTD
Joints in steel construction: moment connections
SCI, BCSA, 1995

5 BRITISH STANDARDS INSTITUTION
BS 5950: Structural use of steelwork in building
Part 3: Design in composite construction
Section 3.1: 1990: Code of practice for design of simple and continuous composite beams
BSI, 1990

6 BRITISH STANDARDS INSTITUTION
BS 5950: Structural use of steelwork in building
Part 1: 1990: Code of practice for design in simple and continuous construction: hot rolled sections
BSI, 1990

7 BRITISH STANDARDS INSTITUTION
DD-ENV 1993-1-1: 1992 Eurocode 3: Design of steel structures
Part 1.1 General rules and rules for buildings
BSI, 1993

8 EUROPEAN COMMISSION
COST C1 Semi-rigid behaviour of civil engineering structural connections
Composite steel-concrete joints in braced frames for buildings
EC, 1996

9 THE BRITISH CONSTRUCTIONAL STEELWORK ASSOCIATION LTD and THE STEEL CONSTRUCTION INSTITUTE
National structural steelwork specification for building construction (3rd edition)
BCSA, 1994

10 BRITISH STANDARDS INSTITUTION
DD-ENV 1994-1-1: 1994 Eurocode 4: Design of composite steel and concrete structures
Part 1.1 General rules and rules for buildings
BSI, 1994

11 BRITISH STANDARDS INSTITUTION
BS 8110: The structural use of concrete
Part 1: 1997: Code of practice for design and construction
BSI, 1997

12 AHMED, B., and NETHERCOT, D. A.
Effect of high shear on the moment capacity of composite cruciform end plate connections
Journal of Constructional Steel Research, Vol. 40, No. 2, November 1996

13 Building Regulations 1991 (1994 edition)
The Stationery Office

14 NEAL, B. G.
The plastic methods of structural analysis
Science Paperbacks and Chapman & Hall, 1965

15 COUCHMAN, G.H., and WAY, A.
Ductility requirements for composite connections
(To be published)

16 WYATT, T.A.
Design guide on the vibration of floors
The Steel Construction Institute and CIRIA, 1989

17 JOHNSON, R.P., and ANDERSON, D.
Designers' handbook to Eurocode 4 Part 1.1: Design of composite steel and concrete structures
Thomas Telford, 1993

18 THE INSTITUTION OF STRUCTURAL ENGINEERS
Manual for the design of steelwork building structures
IstructE, 1989

19 COUCHMAN, G.H.
Design of semi-continuous braced frames
The Steel Construction Institute, 1997

20 BRITISH STANDARDS INSTITUTION
BS 4449: 1988: Specification for carbon steel bars for the reinforcement of concrete
BSI, 1988

21 BRITISH STANDARDS INSTITUTION
BS EN 10080: 1996: Steel for the reinforcement of concrete.
Weldable ribbed reinforcing steel B500.
Technical delivery conditions for bars, coils and welded fabric
BSI, 1996

22 BRITISH STANDARDS INSTITUTION
BS 4466: 1989: Specification for scheduling, dimensioning, bending and cutting of reinforcement for concrete
BSI, 1989

23 LI, T.Q., NETHERCOT, D.A., and CHOO, B. S.
Behaviour of flush end plate composite connections with unbalanced moment and variable shear/moment ratios - II. Prediction of moment capacity
Journal of Constructional Steel Research, Vol. 38, No. 2, 1996

24 BAILEY, J.R.
Strength and rigidity of bolted beam-to-column connections
Proceedings, Conference on structures
University of Sheffield, 1970

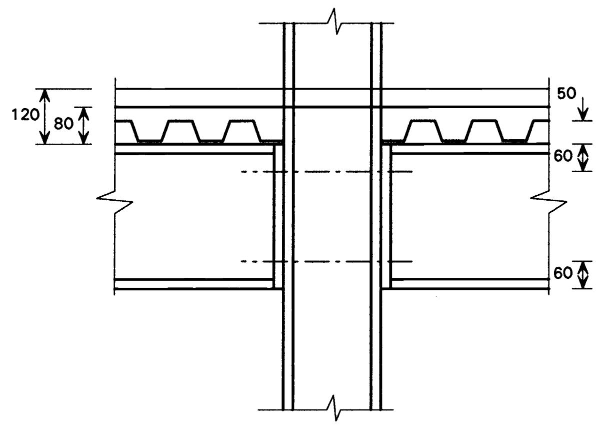
APPENDIX A Worked Example

The following calculation demonstrates the procedure used to calculate the moment capacity of an internal beam to column flange composite connection.

The procedure to allow for interaction between several rows of tension bolts does not differ from that given in Reference 4, and is therefore not included in the worked example. More details may be found in Reference 4.

Contents:

1. CONNECTION DETAILS



- *6 No. 16 mm diameter rebars*
- *1 row M20 8.8 tension bolts*
- *200 × 12 S275 end plate*
- *406 × 178 × 54 UB*
- *203 × 203 × 52 UC*

The beam and column are both S275. The rebar is grade 460B to BS 4449.

Steelwork details are as shown on page 87.

2. TENSION ZONE

2.1 Rebar — *Step 1A*

Potential resistance

For 6 No 16 mm bars, $A_{reinf} = 1210\ mm^2$

530 | 1 *(see worksheet, page 77)*

$$P_{reinf} = \frac{f_y \times A_{reinf}}{\gamma_m} = \frac{460 \times 1210 \times 10^{-3}}{1.05} = 530.1\ kN$$

Check minimum area of reinforcement

Beam is grade S275

From Table 4.1, minimum allowable area of rebar for a 'plastic' connection to an S275, 406 deep beam $= 1150\ mm^2 < 1210\ mm^2$ *OK*

Check maximum area of reinforcement

To prevent concrete crushing:

$$A_L \times \frac{0.6 \times b_c \times d_s}{\mu} \times \frac{f_{cu}}{f_y} = \frac{0.6 \times 204.3 \times (120 - 50)}{\mu} \times \frac{30}{460}$$

For this example, it is assumed that a connection of equal strength will be adopted on the opposing column flange, $\therefore \mu$ = zero, so $A_L \leq \infty$
So A_{reinf} = 1210 mm^2 is OK.

In addition, in a 'balanced' situation, only nominal transverse reinforcement will be required.

2.2 Bolt Row 1 (only row in tension)

Column side geometry (with reference to Figure 4.7): — Step 1B

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8 \times r = \frac{90}{2} - \frac{7.9}{2} - 0.8 \times 10.2 = 32.9 \text{ mm}$$

$$e = \frac{B - g}{2} = \frac{204.3 - 90}{2} = 57.2 \text{ mm}$$

n = *minimum of e, 1.25m or e (beam side, see below)*

= 57.2 or 1.25 × 32.9 or 55 = 41.1 mm

Column flange bending

From Table 4.5, L_{eff} is given by Min {ii,i}

From Table 4.4, Pattern (i):

$2\pi m = 2\pi \times 32.9 = 206.7$ mm

Table 4.4, Pattern (ii):

$4m + 1.25e = 4 \times 32.9 + 1.25 \times 57.2$

Hence L_{eff} = 203.1 mm

M_p for the column flange:

$$M_p = \frac{L_{eff} \times T_c^2 \times p_y}{4} = \frac{203.1 \times 12.5^2 \times 275 \times 10^{-3}}{4}$$

= 2181.7 kNm

Critical failure mode is minimum of:

Mode 1. $$P_r = \frac{4M_p}{m} = \frac{4 \times 2181.7}{32.9} = 265 \text{ kN}$$

Mode 2. $$P_r = \frac{2M_p + n \times \Sigma P_t'}{m + n} = \frac{2 \times 2181.7 + 41.1 \times 2 \times 137}{32.9 + 41.1}$$

= 211.1 kN

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CALCULATION SHEET

Job Composite Moment Connections	**Sheet** 3 of 9
Title Worked example for a bolted end plate	
Client SCI/BCSA Connections Group	
Calcs by GHC / **Checked by** AW	**Date** May 1998

Mode 3. $P_r = \Sigma P_t' = 2 \times 137 = 274$ kN — 211 | 2

Hence $P_r = 211.1$ kN

Column web tension — Step 1C

$L_t = 45 \times 1.73 \times 2 = 155.7$ mm

$P_t = L_t \times t_w \times p_y = 155.7 \times 7.9 \times 275 \times 10^{-3} = 338$ kN — 338 | 3

Beam side geometry: — Step 1B

$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8 \times s_{ww} = \frac{90}{2} - \frac{7.7}{2} - 0.8 \times 8$$
$$= 34.8 \text{ mm}$$

(assume 8FW)

$$e = \frac{b_p - g}{2} = \frac{200}{2} - \frac{90}{2} = 55 \text{ mm}$$

n = minimum of e, 1.25m or e (column side, see above)
$= 57.2$ or 1.25×34.8 or $55 = 43.5$ mm

End plate bending

From tables, L_{eff} is given by:

Min {Max (ii, iii), i} since $g \not> 0.7 B_b$ and $T_b \not< 0.8 t_p$

From Table 4.4 Pattern (ii):

$4m + 1.25e = 4 \times 34.8 + 1.25 \times 55 = 208.0$ mm

Table 4.4 Pattern (iii):

$L_{eff} = \alpha m_1$, where α is obtained from Figure 4.9.

Using:

$m_1 = m = 34.8$ mm

$m_2 = p_1 - T_c - 0.8 \times s_{wf} = 60 - 10.9 - (0.8 \times 10) = 41.1$ mm

$$\lambda_1 = \frac{m_1}{m_1 + e} = \frac{34.8}{34.8 + 55} = 0.39$$

$$\lambda_2 = \frac{m_2}{m_1 + e} = \frac{41.1}{34.8 + 55} = 0.46$$

So from Figure 4.9 $\alpha = 2\pi$

$\alpha m_1 = 218.7$ mm

$\therefore$ Max (ii, iii) = 218.7 mm

Pattern (i):

$2\pi m = 218.7$ mm

Hence $L_{eff} = 218.7$ mm

$$\text{So, } M_p = \frac{L_{eff} \times t_p^2 \times p_y}{4} = \frac{218.7 \times 12^2 \times 275 \times 10^{-3}}{4}$$

$$= 2165.1 \text{ kNm}$$

Critical failure mode is minimum of:

$$\text{Mode 1.} \quad P_r = \frac{4M_p}{m} = \frac{4 \times 2165.1}{34.8} = 249 \text{ kN}$$

$$\text{Mode 2.} \quad P_r = \frac{2M_p + n \times \Sigma P_t'}{m + n} = \frac{2 \times 2165.1 + 43.5 \times 2 \times 137}{34.8 + 43.5} = 208 \text{ kN}$$

$$\text{Mode 3.} \quad P_r = \Sigma P_t' = 2 \times 137 = 274 \text{ kN}$$

208 | 4

Beam web tension

Step 1C

The underside of the beam flange is only 49.1 mm above bolt row 1. The flange is therefore within the web tensile length, so beam web tension can be discounted.

N/A | 5

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CALCULATION SHEET

Job	Sheet
Composite Moment Connections	5 of 9

Title Worked example for a bolted end plate

Client SCI/BCSA Connections Group

Calcs by	Checked by	Date
GHC	AW	May 1998

STEP 1 WORKSHEET: TENSION ZONE

Row	Column Side: Step 1B Flange Bending	Column Side: Step 1C Web Tension	Beam Side: Step 1B Plate Bending	Beam Side: Step 1C Web Tension	Step 1D Modify Bolt Forces	Potential Resistance
Reinforcement						From Step 1A P_{reinf} = 530 [1]
1	Resistance of row 1: 211 [2]	338 [3]	208 [4]	NA [5]	NA [6]	least of boxes 2 to 6 gives P_{r1} = 208 [7]
2	Resistance of row 2 alone: [8]	[9]	[10]	[11]		least of boxes: 8 to 11 and 16 to 20 gives
	Resistance of rows(1 + 2) combined: [12]	[13]	[14]	[15]		
	Deduct box 6: [16]	[17]	[18]	[19]	[20]	P_{r2} = [21]
3	Resistance of row 3 alone: [22]	[23]	[24]	[25]		
	Resistance of combined rows (2 + 3): [26]	[27]	[28]	[29]		
	Deduct box 20: [30]	[31]	[32]	[33]		least of boxes 22 to 25 and 30 to 33 and
	Resistance of combined rows(1 + 2 + 3): [34]	[35]	[36]	[37]		38 to 42 gives
	Deduct sum of boxes 6 & 20: [38]	[39]	[40]	[41]	[42]	P_{r3} = [43]

3. COMPRESSION ZONE

3.1 Column Web Crushing

Step 2A

$b_1 = T_b + 2\,s_{wf} + 2\,t_p = 10.9 + 2 \times 10 + 2 \times 12 = 54.9\text{ mm}$

$n_2 = 2.5[T_c + r] \times 2 = (2.5 \times [12.5 + 10.2]) \times 2 = 113.5\text{ mm}$

$\therefore P_c = (b_1 + n) \times t_c \times p_y = (54.9 + 113.5) \times 7.9 \times 275 \times 10^{-3}$
$= 365.8\text{ kN}$

3.2 Column Web Buckling

Step 2A

$n_1 = 206.2\text{ mm}$

p_c is obtained from Table 27(c) of BS 5950 Part 1 using:

$$\lambda = \frac{2.5d}{t_c} = \frac{2.5 \times 160.8}{7.9} = 50.9$$

For $p_y = 275\text{ N/mm}^2$, $p_c = 219\text{ N/mm}^2$

$\therefore P_c = 451.7\text{ kN}$

Therefore, column compression resistance = 366 kN

3.3 Beam Flange Crushing

Step 2B

The potential resistance of the flange alone is:

$P_c = 1.4p_{yb} \times T_b \times B_b = 1.4 \times 275 \times 10^{-3} \times 10.9 \times 177.7 = 745.7\text{ kN}$

4. COLUMN PANEL SHEAR ZONE

Step 3

For this balanced situation, column panel shear is zero.

5. CALCULATION OF MOMENT CAPACITY

Step 4

At this stage, potential reinforcement and bolt forces may need to be modified to satisfy certain criteria and maintain equilibrium.

5.1 Equilibrium

Total potential tensile force (from reinforcement + bolts) is taken from Step 1 Worksheet Boxes 1 and 7 respectively:

$= P_{reinf} + P_{r1} = 530 + 208^* = 738\text{ kN}$

(* this value may need to be modified when Step 1D is applied)

The potential compression resistance of the beam flange alone

P_c = 746 kN

$\therefore$ *None of the beam web will be subject to compression.*

Other compressive capacities (calculated above) are:

*Column web crushing, 366 kN**

*Column web bucking, 452 kN**

** Neither of these column capacities can be allowed to govern the behaviour of a 'plastic' connection.*

As column web crushing is critical, the column requires compression stiffening, as detailed below. Alternatively, a more economic solution might be to use a heavier column section which does not need stiffening.

5.2 Column Compression Stiffeners — *Step 6A*

Beam flange thickness T_b = 10.9 mm

therefore, try 15 mm thick stiffeners

Let width of each stiffener, b_{sg} = 95 mm (= $6.3t_s$)

with a corner snipe of 12 mm

Buckling resistance of stiffener + column

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

where $A_w = 40\, t_c \times t_c = (40 \times 7.9) \times 7.9 = 2496\ mm^2$

$A_{sg} = 2\, b_{sg} \times t_s = 2 \times 95 \times 15 = 2850\ mm^2$

Derivation of p_c

$$r_y \approx = \sqrt{\left[I_s \big/ (A_w + A_{sg})\right]} = \sqrt{\left[\left(\frac{15 \times 198^3}{12}\right) \Big/ (7.9 \times 316 + 15 \times 2 \times 95)\right]}$$

$= 42.6$ mm

$L = D - 2\,T_c = 206.2 - 2 \times 12.5 = 181.2$ mm

$$\therefore \lambda = \frac{0.7 \times L}{r_y} = \frac{0.7 \times 181.2}{42.6} = 2.98$$

So from BS 5950 Part 1 Table 27(c), for $p_y = 275$ N/mm^2:

$p_c = 275$ N/mm^2

$\therefore P_{c\ buckling} = (A_w + A_{sg}) \times p_c = (2496 + 2850) \times 275 \times 10^{-3}$

$= 1470$ kN

Crushing resistance of stiffener + column

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

where; $A_{sn} = 2b_{sn} \times t_s = 2 \times (95-12) \times 15 = 2490$ mm^2

$b_1 =$ (calculated above) $= 54.9$ mm

$n_2 =$ (calculated above) $= 113.5$ mm

$\therefore P_{c\ crushing} = \{(2490 \times 275) + [(54.9 + 113.5) \times 7.9 \times 275]\} \times 10^{-3}$

$= 1051$ kN

Bearing resistance of stiffener alone

$$P_{c\ bearing} = \frac{A_{sn} \times p_{ys}}{0.8} = \frac{2490 \times 275 \times 10^{-3}}{0.8}$$

$= 856$ kN

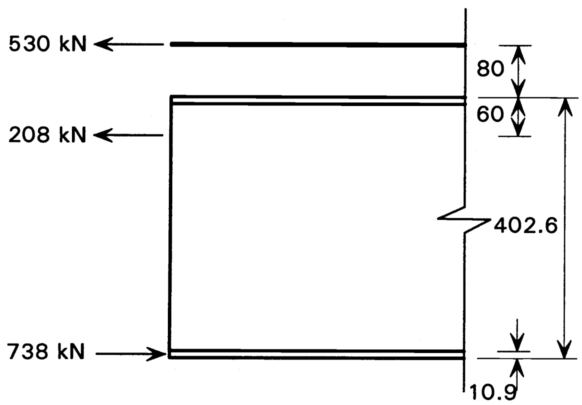
Compression resistance of stiffened column is the minimum of 1470 kN, 1051 kN, and 856 kN

856 kN > 738 kN , so the stiffened column is adequate in compression, OK

5.3 Moment Capacity Step 4

$M_j = (F_{reinf} \times h_{reinf}) + (F_{r1} \times h_{r1})$
$= [530 \times (402.6 + 80 - 10.9/2) + 208 \times (402.6 - 60 -10.9/2)] \times 10^{-3}$

$= 252.9 + 70.1 =$ <u>323 kNm</u>



Note: *The capacity table on page 86 gives a moment capacity of 317 kNm. The difference from the value calculated above is because the tabulated value is based on a conservative estimation of the rebar height. The capacity table also states that rebar must be able to achieve not less than 10% elongation at maximum force for a 'plastic' connection with a 406 mm deep grade S355 beam, and may need this level of ductility with an S275 beam. However, Table 4.1 shows that '10%' rebar is not required in this case.*

6. CALCULATION OF VERTICAL SHEAR CAPACITY — *Step 5*

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

where $n_s = 2$

$P_{ss} =$ *91.9 kN from tables for an M20 bolt*

$n_t = 2$

P_{ts} *is the lesser of:*

36.8 kN for bolt shear (from tables)

110 kN for end plate bearing

115 kN for column flange bearing

$= 36.8$ *kN*

$\therefore$ *Shear capacity* $\approx$ *258 kN* (*as given in the capacity table on page 86*)

APPENDIX B Design Tables for Standard Composite 'Plastic' Connections

B.1 INTRODUCTION

Tables are presented for composite beam-to-column 'plastic' connections, suitable for use in frames designed using the procedures outlined in Section 3. Connections using M20 8.8 bolts, with flush end plate details, are followed by similar details with M24 8.8 bolts. For each steel detail, a range of eight reinforcement options is given. A reinforcement strength of 460 N/mm^2 is assumed.

Tabulated moment capacities are given for beams in design grade S275 or S355, although some restrictions apply (see tables). All end plates are grade S275. Column side capacities for grades S275 and S355 must be checked as described below.

For a connection to work in the intended manner, it is important that the reinforcement details, 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 in Appendix B.

The standard connection capacities are based on the assumption that there is no axial force in the beam.

B.1.1 Beam side

Moment capacity

The moment capacities given for the beam side of the connections were calculated using the procedure specified in Section 4.2. Reinforcement and bolt row forces are included in the tables.

For some details (highlighted with an asterisk *) a 'plastic' connection can only be achieved when reinforcement can attain 10% strain at peak load. In some cases this restriction only applies to S355 beams; reference should be made to Figure 4.7 to check requirements for S275 beams.

Values given in bold indicate that the detail may only be used with S355 beams. This restriction may be necessary either:

- to maintain the neutral axis in the lower half of the steel beam
- to ensure that the connection capacity is less than that of the adjacent beam, thereby ensuring that any plastic hinges will form in the connections rather than at the beam ends.

Dimension A

Dimension 'A' is the lever arm from the centre of compression to the lowest row of tension bolts. It can be used to calculate a reduced moment capacity when tension forces are limited by the column flange strength (see tables).

Weld sizes

All flange welds must be full strength, with a minimum visible fillet of 10 mm. All web welds should be continuous 8 mm fillets. For further information on details of the welds see Section 4.2, Step 7.

B.1.2 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 forces indicated in the beam side table. Where the column has a smaller capacity in flange bending, reduced bolt row forces are shown. A reduced moment capacity may be determined from these lower forces, or the column flange may be stiffened in the tension zone (Section 4.2, Step 6C). If the column capacity is governed by web tension, an S in the table indicates that tension stiffeners (or similar strengthening) are needed to avoid potential non-ductile failure.

The capacities have been calculated assuming that the column top is at least 100 mm above the beam flange or top row of bolts.

Compression zone

A tick ✓ in the table indicates that the column web has a greater compression capacity than the sum of the reinforcement and bolt forces. The check was made assuming a stiff bearing length from the beam side of the connection of 50 mm.

An *S* in the table shows that the column web compression capacity is lower than the sum of the reinforcement and bolt forces. The web must be stiffened to resist these forces (see Section 4.2, Step 6A).

Panel shear zone

The panel shear capacity is that of the column web. The applied web panel shear must take account of beams connecting onto both flanges (see Section 4.2, Step 3).

B.1.3 Worked example using the capacity tables

An example is given below to illustrate the use of the capacity tables. The starting point is some way down the design flow chart given in Figure 3.2.

Worked example using the capacity tables

Choose a composite connection to suit the following member sizes and applied forces:

Column (internal): 254x254x167 UC, grade S355
Beams: 406x178x67 UB, grade S355

Applied moment: 435 kNm
Applied shear: 400 kN

Try a connection adopting one row of M20 bolts, with ten 16 mm reinforcing bars (page 86).

	Beam Side	Column Side
Moment capacity	486kNm (page 86) > 435kNm OK High elongation reinforcement is not needed *[there is no * in the table]* Beam is suitable in S355 only *[capacity is given in bold type]*	Tension zone: ✓ OK Compression zone: ✓ OK (no stiffening required - *there is no S in the table*)
Vertical shear	258kN (page 87) < 400kN Unsatisfactory without optional shear row 442kN (page 87) > 400kN OK with optional shear row	
Column web panel shear		Opposing beams give zero shear across the column web. OK

B.2 CAPACITY TABLES

Contents

Dimensions for detailing are shown on page 98.

1 ROW M20 8.8 BOLTS 200 x 12 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. φ16 804 mm² 351 kN		B 6 No. φ16 1210 mm² 529 kN		C 8 No. φ16 1610 mm² 704 kN		D 10 No. φ16 2010 mm² 878 kN		E 4 No. φ20 1260 mm² 551 kN		F 6 No. φ20 1890 mm² 826 kN		G 8 No. φ20 2510 mm² 1097 kN		H 10 No. φ20 3140 mm² 1372 kN	
	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm
457x191x98	398	268*	398	362*	398	454*	398	546*	398	373*	398	518*	398	*661**	**398**	**807**
89	395	266*	395	359*	395	451*	395	543*	395	371*	395	515*	395	*648**	***395***	***802***
82	392	265*	392	358*	392	449*	392	540*	392	369*	392	513*	**392**	**654***	–	–
74	390	264*	390	356*	390	447*	*384*	*531**	390	367*	*390*	*510**	***390***	***651****	–	–
67	387	262*	387	354*	*387*	*444**	***387***	***535****	387	365*	***387***	***508****	–	–	–	–
457x152x82	396	267*	396	361*	396	453*	*396*	*545**	396	372*	396	517*	**396**	**660***	–	–
74	394	266*	394	359*	394	450*	*382*	*529**	394	370*	*386*	*506**	***387***	***648****	–	–
67	391	264*	391	356*	*385*	*443**	***391***	***538****	391	368*	**391**	**511***	–	–	–	–
60	388	263*	388	355*	***388***	***445****	–	–	*388*	*366**	–	–	–	–	–	–
52	384	261*	**384**	**352***	–	–	–	–	**384**	**363***	–	–	–	–	–	–
406x178x74	345	239*	345	323*	345	406*	*345*	*489*	345	333*	345	464*	***345***	***593***	–	–
67	342	237*	342	321*	*342*	*403**	***336***	***486***	342	331*	**342**	**461***	–	–	–	–
60	340	236*	340	319*	**340**	**401***	–	–	*340*	*330**	***340***	***459****	–	–	–	–
54	337	234*	*337*	*317**	***337***	***399****	–	–	*333*	*324**	–	–	–	–	–	–
406x140x46	338	235*	***338***	***317****	–	–	–	–	***338***	***328****	–	–	–	–	–	–
39	**334**	**232***	–	–	–	–	–	–	–	–	–	–	–	–	–	–
356x171x67	296	211*	296	286*	296	361	*289*	*428*	296	296*	*296*	*413*	***296***	***528***	–	–
57	292	209*	292	284*	**292**	**357**	–	–	292	293*	***292***	***409***	–	–	–	–
51	289	207*	*289*	*282**	***289***	***355***	–	–	*289*	*291**	–	–	–	–	–	–
45	287	206*	***287***	***280****	–	–	–	–	***287***	***289****	–	–	–	–	–	–
356x127x39	*281*	*203**	–	–	–	–	–	–	–	–	–	–	–	–	–	–
33	***280***	***202****	–	–	–	–	–	–	–	–	–	–	–	–	–	–
305x165x54	244	182	244	248	**244**	**313**	–	–	244	256	***244***	***359***	–	–	–	–
46	241	180	*241*	*246*	–	–	–	–	**238**	**254**	–	–	–	–	–	–
40	*238*	*179*	***238***	***244***	–	–	–	–	***238***	***252***	–	–	–	–	–	–
305x127x48	244	182	*237*	*239*	***239***	***308***	–	–	*235*	*244*	–	–	–	–	–	–
42	241	181	*224*	*221*	–	–	–	–	*222*	*225*	–	–	–	–	–	–
37	*232*	*171*	***231***	***233***	–	–	–	–	***229***	***238***	–	–	–	–	–	–
305x102x33	*230*	*163*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
28	*211*	*143*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
25	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
254x146x43	193	151	*188*	*195*	–	–	–	–	*187*	*199*	–	–	–	–	–	–
37	*191*	*149*	*175*	*176*	–	–	–	–	*172*	*179*	–	–	–	–	–	–
31	*177*	*128*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
254x102x28	*172*	*121*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
25	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355
369 Beam must be grade S355 to satisfy neutral axis position requirements
264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)
* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)
256 Connection capacity exceeds 0.8 M_p of composite beam in hogging (see Section 3.2.1 for significance of this)
The value of F_{r1} is based on the assumption that the NA is at least 200 mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

1 ROW M20 8.8 BOLTS 200 x 12 S275 END PLATE	COLUMN SIDE

S275																				S355		
Panel Shear Cap.	Web Compn. Cap.	Tension Zone F_{r1}	Compression Zone Reinforcement option								Column Serial Size	Compression Zone Reinforcement option								Tension Zone F_{r1}	Web Compn. Cap.	Panel Shear Cap.
(kN)	(kN)	(kN)	A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	(kN)	(kN)	(kN)
											356x368											
1000	1141	✓	✓	✓	✓	✓	✓	✓	S	S	x 202	✓	✓	✓	✓	✓	✓	✓	S	✓	1486	1302
849	935	✓	✓	✓	✓	S	✓	S	S	S	x 177	✓	✓	✓	✓	✓	✓	S	S	✓	1217	1105
725	766	✓	✓	✓	S	S	✓	S	S	S	x 153	✓	✓	✓	S	✓	S	S	S	✓	974	944
605	605	✓	✓	S	S	S	S	S	S	S	x 129	✓	✓	S	S	✓	S	S	S	✓	788	787
											305x305											
1037	1432	✓	✓	✓	✓	✓	✓	✓	✓	S	x198	✓	✓	✓	✓	✓	✓	✓	✓	✓	1865	1350
816	1051	✓	✓	✓	✓	S	✓	✓	S	S	x158	✓	✓	✓	✓	✓	✓	✓	S	✓	1368	1062
703	858	✓	✓	✓	S	S	✓	S	S	S	x137	✓	✓	✓	✓	✓	✓	S	S	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	x118	✓	✓	S	S	✓	S	S	S	✓	909	774
503	553	✓	S	S	S	S	S	S	S	S	x97	✓	S	S	S	S	S	S	S	✓	713	649
											254x254											
882	1384	✓	✓	✓	✓	✓	✓	✓	✓	S	x167	✓	✓	✓	✓	✓	✓	✓	✓	✓	1802	1149
685	992	✓	✓	✓	✓	S	✓	S	S	S	x132	✓	✓	✓	✓	✓	✓	S	S	✓	1292	892
551	744	✓	✓	✓	S	S	S	S	S	S	x107	✓	✓	✓	S	✓	S	S	S	✓	969	717
434	557	✓	S	S	S	S	S	S	S	S	x89	✓	S	S	S	S	S	S	S	✓	725	566
360	436	✓	S	S	S	S	S	S	S	S	x73	✓	S	S	S	S	S	S	S	✓	563	465
											203x203											
459	701	✓	✓	S	S	S	S	S	S	S	x86	✓	✓	✓	S	✓	S	S	S	✓	913	598
353	512	✓	S	S	S	S	S	S	S	S	x71	✓	S	S	S	S	S	S	S	✓	666	460
322	440	✓	S	S	S	S	S	S	S	S	x60	✓	S	S	S	S	S	S	S	✓	568	415
272	360	✓	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	✓	464	351
245	313	198	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	✓	404	316

Tension Zone:

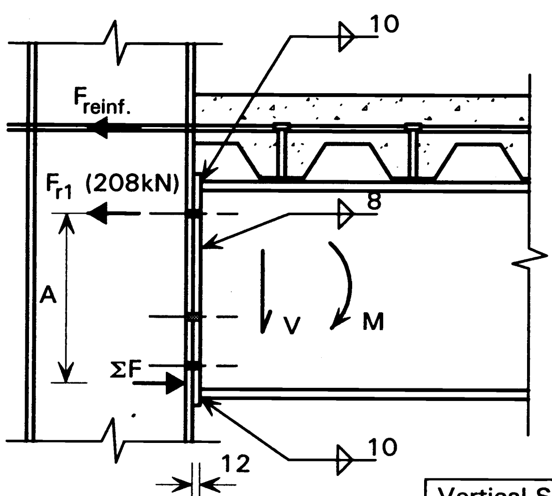
✓ Column satisfactory for bolt row tension values shown for the beam side.

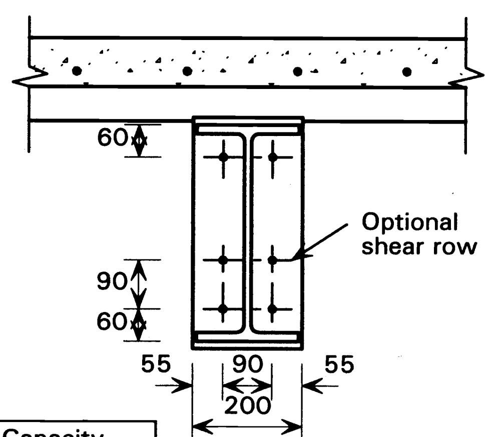
195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

S Provide compression stiffener.





Vertical Shear Capacity
258 kN without shear row
442 kN with shear row

2 ROWS M20 8.8 BOLTS 200 x 12 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. ϕ16 804 mm² 351 kN		B 6 No. ϕ16 1210 mm² 529 kN		C 8 No. ϕ16 1610 mm² 704 kN		D 10 No. ϕ16 2010 mm² 878 kN		E 4 No. ϕ20 1260 mm² 551 kN		F 6 No. ϕ20 1890 mm² 826 kN		G 8 No. ϕ20 2510 mm² 1097 kN		H 10 No. ϕ20 3140 mm² 1372 kN	
	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm
533x210x122	384	363*	384	470*	384	575*	384	681*	384	483*	384	649*	384	813*	*377*	*967**
109	380	360*	380	466*	380	571*	380	676*	380	480*	380	645*	380	807*	**380**	**973***
101	378	358*	378	465*	378	569*	378	674*	378	478*	378	642*	*378*	*791**	***378***	***969****
92	375	357*	375	462*	375	566*	*375*	*670**	375	475*	375	639*	**375**	**800***	–	–
82	372	354*	372	459*	372	563*	**372**	**666***	372	472*	*361*	*623**	–	–	–	–
457x191x98	308	310*	308	403*	308	495*	308	588*	308	415*	308	560*	*301*	*693**	***308***	***848***
89	305	307*	305	401*	305	492*	*305*	*584**	305	412*	305	556*	**305**	**699***	–	–
82	302	306*	302	398*	302	490*	*302*	*572**	302	410*	*302*	*553**	***302***	***695****	–	–
74	300	304*	300	396*	*300*	*487**	**300**	**578***	300	408*	**300**	**551***	–	–	–	–
67	297	302*	297	394*	**297**	**484***	–	–	297	405*	***297***	***548****	–	–	–	–
457x152x82	306	309*	306	402*	306	494*	*294*	*571**	306	414*	*297*	*548**	***301***	***693****	–	–
74	304	307*	304	400*	*295*	*482**	**304**	**583***	304	411*	*285*	*531**	–	–	–	–
67	301	305*	301	397*	*285*	*472**	***293***	***570****	296	404*	***295***	***546****	–	–	–	–
60	298	303*	*288*	*386**	***292***	***479****	–	–	*286*	*395**	–	–	–	–	–	–
52	287	295*	***289***	***387****	–	–	–	–	***287***	***397****	–	–	–	–	–	–
406x178x74	255	273	255	357*	255	440*	*243*	*502*	255	368*	*247*	*485**	***250***	***618***	–	–
67	252	271*	252	355*	*244*	*425**	***252***	***520***	252	365*	***252***	***495****	–	–	–	–
60	250	270*	*250*	*353**	***250***	***435****	–	–	*245*	*357**	–	–	–	–	–	–
54	247	268*	*233*	*331**	–	–	–	–	**247**	**361***	–	–	–	–	–	–
406x140x46	*237*	*255**	–	–	–	–	–	–	–	–	–	–	–	–	–	–
39	***235***	***255****	–	–	–	–	–	–	–	–	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355

369 Beam must be grade S355 to satisfy neutral axis position requirements

264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)

* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)

256 Connection capacity exceeds 0.8*Mp* of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of F_{r1} is based on the assumption that the NA is at least 200mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

2 ROWS M20 8.8 BOLTS 200 x 12 S275 END PLATE	COLUMN SIDE

S275												Column Serial Size	S355											
Panel Shear Cap.	Web Compn. Cap.	Tension Zone		Compression Zone									Compression Zone								Tension Zone		Web Compn. Cap.	Panel Shear Cap.
		F_{r1}	F_{r2}	Reinforcement option									Reinforcement option								F_{r1}	F_{r2}		
(kN)	(kN)	(kN)		A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	(kN)		(kN)	(kN)
												356x368												
1000	1141	✓	✓	✓	✓	✓	S	✓	S	S	S	x 202	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1486	1302
849	935	✓	✓	✓	✓	S	S	✓	S	S	S	x 177	✓	✓	✓	S	✓	✓	S	S	✓	✓	1217	1105
725	766	✓	✓	✓	S	S	S	S	S	S	S	x 153	✓	✓	S	S	✓	S	S	S	✓	✓	974	944
605	605	✓	✓	S	S	S	S	S	S	S	S	x 129	✓	S	S	S	S	S	S	S	✓	✓	788	787
												305x305												
1037	1432	✓	✓	✓	✓	✓	✓	✓	✓	S	S	x198	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	1865	1350
816	1051	✓	✓	✓	✓	✓	S	✓	S	S	S	x158	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1368	1062
703	858	✓	✓	✓	S	S	S	S	S	S	S	x137	✓	✓	✓	S	✓	S	S	S	✓	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	S	x118	✓	✓	S	S	✓	S	S	S	✓	✓	909	774
503	553	✓	✓	S	S	S	S	S	S	S	S	x97	✓	S	S	S	S	S	S	S	✓	✓	713	649
												254x254												
882	1384	✓	✓	✓	✓	✓	✓	✓	✓	S	S	x167	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	1802	1149
685	992	✓	✓	✓	✓	S	S	✓	S	S	S	x132	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1292	892
551	744	✓	✓	✓	S	S	S	S	S	S	S	x107	✓	✓	S	S	✓	S	S	S	✓	✓	969	717
434	557	✓	✓	S	S	S	S	S	S	S	S	x89	✓	S	S	S	S	S	S	S	✓	✓	725	566
360	436	✓	✓	S	S	S	S	S	S	S	S	x73	S	S	S	S	S	S	S	S	✓	✓	563	465
												203x203												
459	701	✓	✓	✓	S	S	S	S	S	S	S	x86	✓	✓	S	S	✓	S	S	S	✓	✓	913	598
353	512	✓	✓	S	S	S	S	S	S	S	S	x71	S	S	S	S	S	S	S	S	✓	✓	666	460
322	440	✓	✓	S	S	S	S	S	S	S	S	x60	S	S	S	S	S	S	S	S	✓	✓	568	415
272	360	✓	✓	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	✓	✓	464	351
245	313	198	97	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	✓	✓	404	316

Tension Zone:

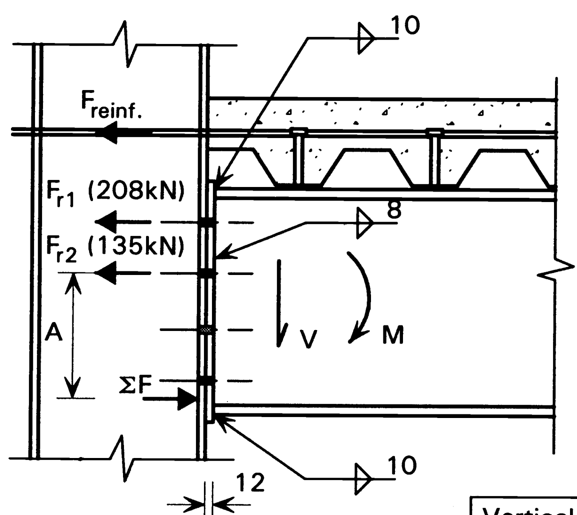
✓ Column satisfactory for bolt row tension values shown for the beam side.

195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

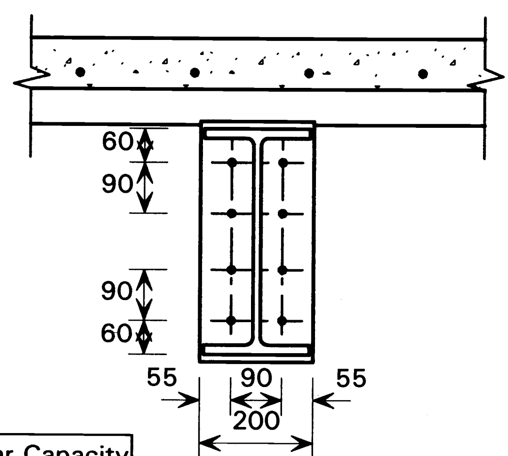
Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

S Provide compression stiffener.



Vertical Shear Capacity
515 kN



2 ROWS M20 8.8 BOLTS 250 x 12 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. φ16 804 mm² 351 kN		B 6 No. φ16 1210 mm² 529 kN		C 8 No. φ16 1610 mm² 704 kN		D 10 No. φ16 2010 mm² 878 kN		E 4 No. φ20 1260 mm² 551 kN		F 6 No. φ20 1890 mm² 826 kN		G 8 No. φ20 2510 mm² 1097 kN		H 10 No. φ20 3140 mm² 1372 kN	
	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm
533x210x122	384	375*	384	482*	384	588*	384	693*	384	495*	384	662*	384	825*	*375*	*976**
109	380	372*	380	479*	380	583*	380	688*	380	492*	380	657*	375	811*	**380**	**985***
101	378	371*	378	477*	378	581*	378	686*	378	490*	378	654*	*367*	*800**	***378***	***981****
92	375	369*	375	474*	375	578*	*375*	*682**	375	487*	375	651*	***375***	***812****	–	–
82	372	366*	372	471*	366	569*	**372**	**678***	372	484*	*359*	*631**	–	–	–	–
457x191x98	308	319*	308	413*	308	505*	308	597*	308	425*	308	570*	*299*	*700**	***308***	***858***
89	305	317*	305	410*	305	502*	*305*	*594**	305	422*	305	566*	**305**	**708***	–	–
82	305	315*	302	408*	302	499*	*293*	*579**	302	419*	*296*	*556**	***302***	***705****	–	–
74	300	314*	300	406*	*294*	*491**	***300***	***588****	300	417*	**300**	**560***	–	–	–	–
67	297	312*	297	404*	**297**	**494***	–	–	*297*	*415**	***293***	***552****	–	–	–	–
457x152x82	306	319*	306	412*	306	504*	*292*	*578**	306	423*	*295*	*555**	***300***	***701****	–	–
74	304	316*	304	409*	*293*	*489**	**304**	**592***	304	421*	*283*	*534**	–	–	–	–
67	301	314*	296	402*	*283*	*475**	***291***	***577****	294	412*	***294***	***553****	–	–	–	–
60	298	312*	*285*	*393**	–	–	–	–	**298**	**416***	–	–	–	–	–	–
52	284	303*	***287***	***394****	–	–	–	–	***285***	***404****	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355

369 Beam must be grade S355 to satisfy neutral axis position requirements

264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)

* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)

256 Connection capacity exceeds 0.8*M*p of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of F_{r1} is based on the assumption that the NA is at least 200mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

2 ROWS M20 8.8 BOLTS
250 x 12 S275 END PLATE

COLUMN SIDE

S275 Panel Shear Cap. (kN)	S275 Web Compn. Cap. (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Compression Zone A	B	C	D	E	F	G	H	Column Serial Size	S355 Compression Zone A	B	C	D	E	F	G	H	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Web Compn. Cap. (kN)	S355 Panel Shear Cap. (kN)
												356x368												
1000	1141	✓	✓	✓	✓	✓	S	✓	S	S	S	x 202	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1486	1302
849	935	✓	✓	✓	✓	S	S	✓	S	S	S	x 177	✓	✓	✓	S	✓	✓	S	S	✓	✓	1217	1105
725	766	✓	✓	✓	S	S	S	S	S	S	S	x 153	✓	✓	S	S	✓	S	S	S	✓	✓	974	944
605	605	✓	✓	S	S	S	S	S	S	S	S	x 129	✓	S	S	S	S	S	S	S	✓	✓	788	787
												305x305												
1037	1432	✓	✓	✓	✓	✓	✓	✓	✓	S	S	x198	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	1865	1350
816	1051	✓	✓	✓	✓	S	S	✓	S	S	S	x158	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1368	1062
703	858	✓	✓	✓	S	S	S	S	S	S	S	x137	✓	✓	✓	S	✓	S	S	S	✓	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	S	x118	✓	✓	S	S	S	S	S	S	✓	✓	909	774
503	553	✓	✓	S	S	S	S	S	S	S	S	x97	S	S	S	S	S	S	S	S	✓	✓	713	649
												254x254												
882	1384	✓	✓	✓	✓	✓	✓	✓	✓	S	S	x167	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	1802	1149
685	992	✓	✓	✓	✓	S	S	✓	S	S	S	x132	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1292	892
551	744	✓	✓	✓	S	S	S	S	S	S	S	x107	✓	✓	S	S	✓	S	S	S	✓	✓	969	717
434	557	✓	✓	S	S	S	S	S	S	S	S	x89	S	S	S	S	S	S	S	S	✓	✓	725	566
360	436	✓	✓	S	S	S	S	S	S	S	S	x73	S	S	S	S	S	S	S	S	✓	✓	563	465
												203x203												
459	701	✓	✓	S	S	S	S	S	S	S	S	x86	✓	✓	S	S	S	S	S	S	✓	✓	913	598
353	512	✓	✓	S	S	S	S	S	S	S	S	x71	S	S	S	S	S	S	S	S	✓	✓	666	460
322	440	✓	✓	S	S	S	S	S	S	S	S	x60	S	S	S	S	S	S	S	S	✓	✓	568	415
272	360	✓	✓	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	✓	✓	464	351
245	313	198	97	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	✓	✓	404	316

Tension Zone:

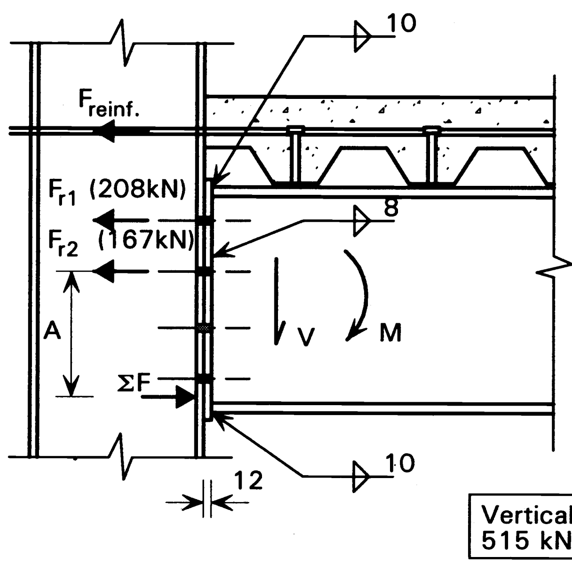
✓ Column satisfactory for bolt row tension values shown for the beam side.

195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

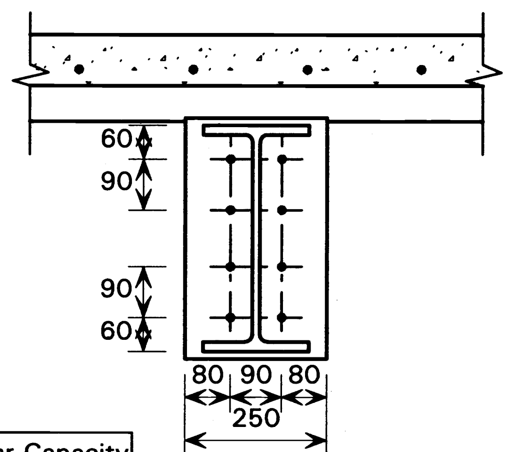
Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

S Provide compression stiffener.



Vertical Shear Capacity
515 kN



1 ROW M24 8.8 BOLTS 200 x 15 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. ϕ16 804 mm² 351 kN		B 6 No. ϕ16 1210 mm² 529 kN		C 8 No. ϕ16 1610 mm² 704 kN		D 10 No. ϕ16 2010 mm² 878 kN		E 4 No. ϕ20 1260 mm² 551 kN		F 6 No. ϕ20 1890 mm² 826 kN		G 8 No. ϕ20 2510 mm² 1097 kN		H 10 No. ϕ20 3140 mm² 1372 kN	
	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm
457x191x98	398	307*	398	401*	398	493*	398	585*	398	412*	398	557*	*392*	*693**	***398***	***846***
89	395	305*	395	398*	395	490*	395	581*	395	410*	395	554*	**395**	**696***	–	–
82	392	303*	392	396*	392	487*	*387*	*572**	392	407*	*392*	*551**	***392***	***693****	–	–
74	390	302*	390	394*	*390*	*485**	***390***	***576****	390	405*	**390**	**549***	–	–	–	–
67	387	300*	387	392*	**387**	**482***	–	–	387	403*	***387***	***545****	–	–	–	–
457x152x82	396	306*	396	400*	396	492*	*386*	*572**	396	411*	*389*	*548**	***396***	***699****	–	–
74	394	304*	394	397*	*387*	*482**	**394**	**580***	394	409*	*378*	*536**	–	–	–	–
67	391	302*	391	395*	*378*	*473**	***385***	***570****	391	406*	***391***	***549****	–	–	–	–
60	388	301*	*381*	*386**	***388***	***483****	–	–	*379*	*396**	–	–	–	–	–	–
52	380	295*	***381***	***386****	–	–	–	–	***379***	***397****	–	–	–	–	–	–
406x178x74	345	272*	345	357*	345	440*	*336*	*512*	345	367*	*339*	*491**	***345***	***626***	–	–
67	342	271*	342	354*	*336*	*431**	***342***	***520***	342	365*	***342***	***495****	–	–	–	–
60	340	269*	*340*	*353**	***340***	***435****	–	–	*340*	*363**	–	–	–	–	–	–
54	337	267*	**337**	**350***	–	–	–	–	**337**	**360***	–	–	–	–	–	–
406x140x46	*331*	*263**	–	–	–	–	–	–	–	–	–	–	–	–	–	–
39	*327*	*261**	–	–	–	–	–	–	–	–	–	–	–	–	–	–
356x171x67	296	240*	296	315*	*296*	*390*	**296**	**464**	296	325*	**296**	**442**	–	–	–	–
57	292	237*	*292*	*312**	***292***	***386***	–	–	*292*	*321**	–	–	–	–	–	–
51	289	236*	**289**	**310***	–	–	–	–	***289***	***319****	–	–	–	–	–	–
45	*282*	*231**	–	–	–	–	–	–	–	–	–	–	–	–	–	–
356x127x39	***273***	***235****	–	–	–	–	–	–	–	–	–	–	–	–	–	–
33	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
305x165x54	244	206	*244*	*272*	***244***	***337***	–	–	*244*	*280*	–	–	–	–	–	–
46	241	204	***241***	***270***	–	–	–	–	***241***	***278***	–	–	–	–	–	–
40	*234*	*193*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
305x127x48	244	206	*229*	*242*	–	–	–	–	*227*	*246*	–	–	–	–	–	–
42	*232*	*189*	*213*	*216*	–	–	–	–	*210*	*219*	–	–	–	–	–	–
37	*221*	*170*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
305x102x33	*215*	*157*	–	–	–	–	–	–	–	–	–	–	–	–	–	–
28	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
25	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
254x146x43	*193*	*169*	*180*	*190*	–	–	–	–	*177*	*193*	–	–	–	–	–	–
37	183	144	–	–	–	–	–	–	–	–	–	–	–	–	–	–
31	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355

369 Beam must be grade S355 to satisfy neutral axis position requirements

264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)

* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)

256 Connection capacity exceeds 0.8Mp of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of F_{r1} is based on the assumption that the NA is at least 200mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

1 ROW M24 8.8 BOLTS 200 x 12 S275 END PLATE	COLUMN SIDE

S275											Column Serial Size	S355										
Panel Shear Cap.	Web Compn. Cap.	Tension Zone F_{r1}	Compression Zone – Reinforcement option									Compression Zone – Reinforcement option								Tension Zone F_{r1}	Web Compn. Cap.	Panel Shear Cap.
(kN)	(kN)	(kN)	A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	(kN)	(kN)	(kN)
											356x368											
1000	1141	✓	✓	✓	✓	S	✓	✓	S	S	x 202	✓	✓	✓	✓	✓	✓	✓	S	✓	1486	1302
849	935	✓	✓	✓	S	S	✓	S	S	S	x 177	✓	✓	✓	✓	✓	✓	S	S	✓	1217	1105
725	766	✓	✓	S	S	S	S	S	S	S	x 153	✓	✓	S	S	✓	S	S	S	✓	974	944
605	605	✓	S	S	S	S	S	S	S	S	x 129	✓	S	S	S	S	S	S	S	✓	788	787
											305x305											
1037	1432	✓	✓	✓	✓	✓	✓	✓	✓	S	x198	✓	✓	✓	✓	✓	✓	✓	✓	✓	1865	1350
816	1051	✓	✓	✓	✓	S	✓	S	S	S	x158	✓	✓	✓	✓	✓	✓	S	S	✓	1368	1062
703	858	✓	✓	✓	S	S	✓	S	S	S	x137	✓	✓	✓	S	✓	S	S	S	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	x118	✓	✓	S	S	✓	S	S	S	✓	909	774
503	553	✓	S	S	S	S	S	S	S	S	x97	✓	S	S	S	S	S	S	S	✓	713	649
											254x254											
882	1384	✓	✓	✓	✓	✓	✓	✓	S	S	x167	✓	✓	✓	✓	✓	✓	✓	✓	✓	1802	1149
685	992	✓	✓	✓	S	S	✓	S	S	S	x132	✓	✓	✓	✓	✓	✓	S	S	✓	1292	892
551	744	✓	✓	S	S	S	S	S	S	S	x107	✓	✓	S	S	✓	S	S	S	✓	969	717
434	557	✓	S	S	S	S	S	S	S	S	x89	✓	S	S	S	S	S	S	S	✓	725	566
360	436	297	S	S	S	S	S	S	S	S	x73	S	S	S	S	S	S	S	S	✓	563	465
											203x203											
459	701	✓	✓	S	S	S	S	S	S	S	x86	✓	✓	S	S	✓	S	S	S	✓	913	598
353	512	✓	S	S	S	S	S	S	S	S	x71	✓	S	S	S	S	S	S	S	✓	666	460
322	440	297	S	S	S	S	S	S	S	S	x60	S	S	S	S	S	S	S	S	✓	568	415
272	360	265	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	296	464	351
245	313	204	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	263	404	316

Tension Zone:

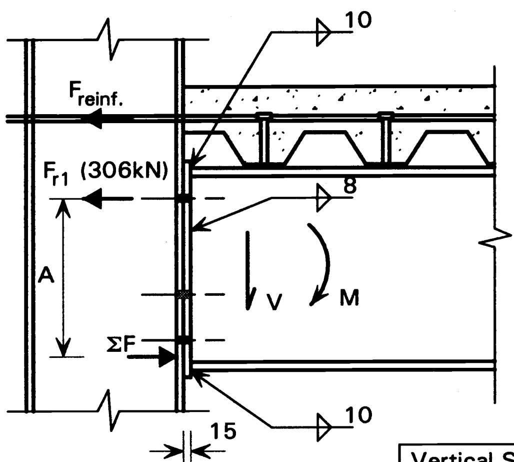
✓ Column satisfactory for bolt row tension values shown for the beam side.

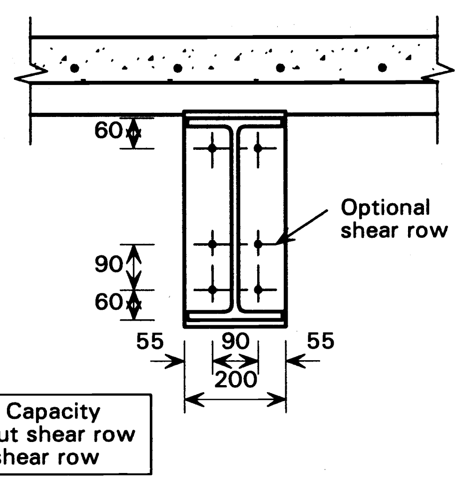
195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

S Provide compression stiffener.





2 ROWS M24 8.8 BOLTS 200 x 15 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. ϕ16 804 mm² 351 kN		B 6 No. ϕ16 1210 mm² 529 kN		C 8 No. ϕ16 1610 mm² 704 kN		D 10 No. ϕ16 2010 mm² 878 kN		E 4 No. ϕ20 1260 mm² 551 kN		F 6 No. ϕ20 1890 mm² 826 kN		G 8 No. ϕ20 2510 mm² 1097 kN		H 10 No. ϕ20 3140 mm² 1372 kN	
	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm	'A' mm	M_C kNm
533x210x122	384	445*	384	552*	384	658*	384	763*	384	566*	384	732*	384	895*	_367_	_1029*_
109	380	442*	380	548*	380	653*	380	758*	380	561*	380	727*	_366_	_867*_	___380___	___1054*___
101	378	440*	378	546*	378	651*	_370_	_744*_	378	559*	373	716*	__378__	__886*__	–	–
92	375	437*	375	543*	_375_	_647*_	_362_	_732*_	375	556*	_365_	_706*_	___370___	___873*___	–	–
82	372	434*	367	534*	__372__	__643*__	___366___	___738*___	366	546*	___372___	___715*___	–	–	–	–
457x191x98	308	377*	308	471*	308	563*	_302_	_647*_	308	483*	_308_	_628*_	_289_	_734*_	___299___	___900___
89	305	375*	305	468*	305	560*	_291_	_631*_	305	479*	_294_	_610*_	___305___	___766*___	–	–
82	302	373*	302	465*	_294_	_547*_	_282_	_610*_	302	477*	_286_	_594*_	–	–	–	–
74	300	371*	295	458*	__300__	__554*__	___293___	___635*___	294	468*	___300___	___617*___	–	–	–	–
67	297	368*	_283_	_444*_	___291___	___543*___	–	–	_282_	_450*_	–	–	–	–	–	–
457x152x82	306	376*	306	470*	_293_	_545*_	_279_	_600*_	306	481*	_283_	_585*_	___292___	___745*___	–	–
74	304	374*	_294_	_456*_	_279_	_516*_	___292___	___634*___	_292_	_466*_	_267_	_549*_	–	–	–	–
67	301	371*	_284_	_443*_	___292___	___544*___	–	–	_282_	_450*_	–	–	–	–	–	–
60	287	_359*_	___291___	___454*___	–	–	–	–	___290___	___464*___	–	–	–	–	–	–
52	267	_327*_	–	–	–	–	–	–	–	–	–	–	–	–	–	–
406x178x74	255	331*	255	415*	_242_	_469*_	_228_	_510_	255	425*	_232_	_503*_	–	–	–	–
67	252	328*	_243_	_394*_	_228_	_441*_	–	–	_241_	_400*_	___244___	___533*___	–	–	–	–
60	245	320*	_231_	_369*_	–	–	–	–	_228_	_374*_	–	–	–	–	–	–
54	_232_	_296*_	_212_	_322*_	–	–	–	–	_209_	_323*_	–	–	–	–	–	–
406x140x46	_213_	_251*_	–	–	–	–	–	–	–	–	–	–	–	–	–	–
39	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355

__369__ Beam must be grade S355 to satisfy neutral axis position requirements

264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)

* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)

_256_ Connection capacity exceeds 0.8M_p of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of F_{r1} is based on the assumption that the NA is at least 200 mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

2 ROWS M24 8.8 BOLTS 200 x 15 S275 END PLATE	COLUMN SIDE

S275												Column Serial Size	S355											
Panel Shear Cap.	Web Compn. Cap.	Tension Zone		Compression Zone									Compression Zone								Tension Zone		Web Compn. Cap.	Panel Shear Cap.
		F_{r1}	F_{r2}	Reinforcement option									Reinforcement option								F_{r1}	F_{r2}		
(kN)	(kN)	(kN)		A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	(kN)		(kN)	(kN)
												356x368												
1000	1141	✓	✓	✓	✓	S	S	✓	S	S	S	x 202	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1486	1302
849	935	✓	✓	✓	S	S	S	S	S	S	S	x 177	✓	✓	S	S	✓	S	S	S	✓	✓	1217	1105
725	766	✓	✓	S	S	S	S	S	S	S	S	x 153	✓	S	S	S	S	S	S	S	✓	✓	974	944
605	605	✓	✓	S	S	S	S	S	S	S	S	x 129	S	S	S	S	S	S	S	S	✓	✓	788	787
												305x305												
1037	1432	✓	✓	✓	✓	✓	✓	✓	✓	S	S	x198	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1865	1350
816	1051	✓	✓	✓	S	S	S	S	S	S	S	x158	✓	✓	✓	S	✓	✓	S	S	✓	✓	1368	1062
703	858	✓	✓	S	S	S	S	S	S	S	S	x137	✓	✓	S	S	✓	S	S	S	✓	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	S	x118	✓	S	S	S	S	S	S	S	✓	✓	909	774
503	553	✓	✓	S	S	S	S	S	S	S	S	x97	S	S	S	S	S	S	S	S	✓	✓	713	649
												254x254												
882	1384	✓	✓	✓	✓	✓	S	✓	✓	S	S	x167	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1802	1149
685	992	✓	✓	✓	S	S	S	S	S	S	S	x132	✓	✓	✓	S	✓	S	S	S	✓	✓	1292	892
551	744	✓	✓	S	S	S	S	S	S	S	S	x107	✓	S	S	S	S	S	S	S	✓	✓	969	717
434	557	✓	✓	S	S	S	S	S	S	S	S	x89	S	S	S	S	S	S	S	S	✓	✓	725	566
360	436	297	✓	S	S	S	S	S	S	S	S	x73	S	S	S	S	S	S	S	S	✓	✓	563	465
												203x203												
459	701	✓	✓	S	S	S	S	S	S	S	S	x86	✓	S	S	S	S	S	S	S	✓	✓	913	598
353	512	✓	✓	S	S	S	S	S	S	S	S	x71	S	S	S	S	S	S	S	S	✓	✓	666	460
322	440	297	204	S	S	S	S	S	S	S	S	x60	S	S	S	S	S	S	S	S	✓	✓	568	415
272	360	265	118	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	296	198	464	351
245	313	204	90	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	263	116	404	316

Tension Zone:

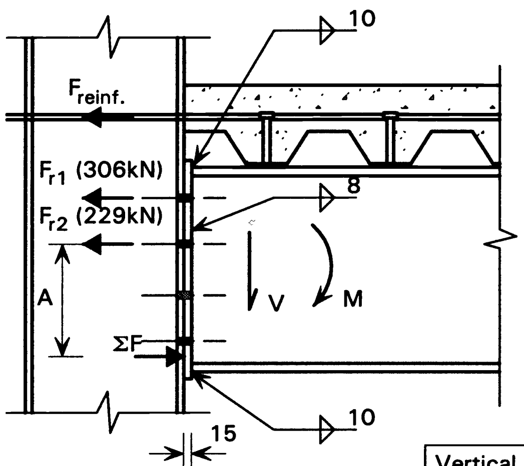
✓ Column satisfactory for bolt row tension values shown for the beam side.

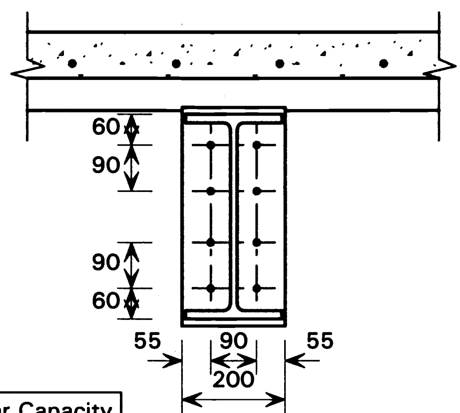
195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

S Provide compression stiffener.





Vertical Shear Capacity
739 kN

2 ROWS M24 8.8 BOLTS 250 x 15 S275 END PLATE	BEAM SIDE

BEAM Serial Size	Effective reinforcement (option, number and size of bars, A_{reinf}, F_{reinf})															
	A 4 No. ϕ16 804 mm^2 351 kN		B 6 No. ϕ16 1210 mm^2 529 kN		C 8 No. ϕ16 1610 mm^2 704 kN		D 10 No. ϕ16 2010 mm^2 878 kN		E 4 No. ϕ20 1260 mm^2 551 kN		F 6 No. ϕ20 1890 mm^2 826 kN		G 8 No. ϕ20 2510 mm^2 1097 kN		H 10 No. ϕ20 3140 mm^2 1372 kN	
	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm	'A' mm	M_c kNm
533x210x122	384	459*	384	566*	384	671*	384	777*	384	579*	384	745*	384	909*	365	1038*
109	380	455*	380	562*	380	666*	380	771*	380	575*	380	740*	364	876*	374	1055*
101	378	453*	378	559*	378	664*	368	754*	378	572*	371	727*	378	899*	–	–
92	375	451*	375	556*	370	653*	359	741*	375	569*	363	716*	369	884*	–	–
82	372	447*	365	545*	353	632*	364	748*	364	557*	343	721*	–	–	–	–
457x191x98	308	388*	308	482*	308	574*	300	655*	308	493*	308	639*	287	736*	298	908
89	305	385*	305	478*	299	563*	288	633*	305	490*	292	617*	299	766*	–	–
82	302	383*	302	476*	292	554*	279	610*	302	487*	283	596*	–	–	–	–
74	300	381*	293	466*	280	529*	292	643*	292	476*	294	620*	–	–	–	–
67	297	379*	280	446*	289	551*	–	–	278	451*	–	–	–	–	–	–
457x152x82	306	387*	301	475*	290	550*	276	599*	300	485*	280	585*	–	–	–	–
74	304	384*	291	464*	276	516*	290	641*	289	473*	263	546*	–	–	–	–
67	295	376*	281	445*	290	552*	–	–	279	451*	–	–	–	–	–	–
60	283	365*	289	462*	–	–	–	–	288	472*	–	–	–	–	–	–
52	262	326*	–	–	–	–	–	–	–	–	–	–	–	–	–	–

398 Beam may be either grade S275 or grade S355

369 Beam must be grade S355 to satisfy neutral axis position requirements

264 Beam must be grade S275 to satisfy minimum reinforcement requirements (see Table 4.1)

* Reinforcement requires a guaranteed strain at maximum load of at least 10% for S355 beams, and possibly for S275 beams (check using Table 4.1)

256 Connection capacity exceeds 0.8 Mp of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of F_{r1} is based on the assumption that the NA is at least 200 mm below the bolt row. It should be reduced in accordance with Section 4.2 Step 1D when necessary.

2 ROWS M24 8.8 BOLTS 250 x 15 S275 END PLATE	COLUMN SIDE

S275												Column Serial Size	S355											
Panel Shear Cap.	Web Compn. Cap.	Tension Zone		Compression Zone									Compression Zone								Tension Zone		Web Compn. Cap.	Panel Shear Cap.
		F_{r1}	F_{r2}	Reinforcement option									Reinforcement option								F_{r1}	F_{r2}		
(kN)	(kN)	(kN)		A	B	C	D	E	F	G	H		A	B	C	D	E	F	G	H	(kN)		(kN)	(kN)
												356x368												
1000	1141	✓	✓	✓	✓	S	S	✓	S	S	S	x 202	✓	✓	✓	✓	✓	✓	S	S	✓	✓	1486	1302
849	935	✓	✓	✓	S	S	S	S	S	S	S	x 177	✓	✓	S	S	✓	S	S	S	✓	✓	1217	1105
725	766	✓	✓	S	S	S	S	S	S	S	S	x 153	✓	S	S	S	S	S	S	S	✓	✓	974	944
605	605	✓	✓	S	S	S	S	S	S	S	S	x 129	S	S	S	S	S	S	S	S	✓	✓	788	787
												305x305												
1037	1432	✓	✓	✓	✓	✓	S	✓	✓	S	S	x198	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1865	1350
816	1051	✓	✓	✓	S	S	S	S	S	S	S	x158	✓	✓	✓	S	✓	S	S	S	✓	✓	1368	1062
703	858	✓	✓	S	S	S	S	S	S	S	S	x137	✓	✓	S	S	S	S	S	S	✓	✓	1116	915
595	692	✓	✓	S	S	S	S	S	S	S	S	x118	S	S	S	S	S	S	S	S	✓	✓	909	774
503	553	✓	✓	S	S	S	S	S	S	S	S	x97	S	S	S	S	S	S	S	S	✓	✓	713	649
												254x254												
882	1384	✓	✓	✓	✓	✓	S	✓	S	S	S	x167	✓	✓	✓	✓	✓	✓	✓	S	✓	✓	1802	1149
685	992	✓	✓	✓	S	S	S	S	S	S	S	x132	✓	✓	✓	S	✓	S	S	S	✓	✓	1292	892
551	744	✓	✓	S	S	S	S	S	S	S	S	x107	✓	S	S	S	S	S	S	S	✓	✓	969	717
434	557	✓	✓	S	S	S	S	S	S	S	S	x89	S	S	S	S	S	S	S	S	✓	✓	725	566
360	436	297	✓	S	S	S	S	S	S	S	S	x73	S	S	S	S	S	S	S	S	✓	✓	563	465
												203x203												
459	701	✓	✓	S	S	S	S	S	S	S	S	x86	S	S	S	S	S	S	S	S	✓	✓	913	598
353	512	✓	✓	S	S	S	S	S	S	S	S	x71	S	S	S	S	S	S	S	S	✓	✓	666	460
322	440	297	204	S	S	S	S	S	S	S	S	x60	S	S	S	S	S	S	S	S	✓	✓	568	415
272	360	265	118	S	S	S	S	S	S	S	S	x52	S	S	S	S	S	S	S	S	296	198	464	351
245	313	204	90	S	S	S	S	S	S	S	S	x46	S	S	S	S	S	S	S	S	263	116	404	316

Tension Zone:

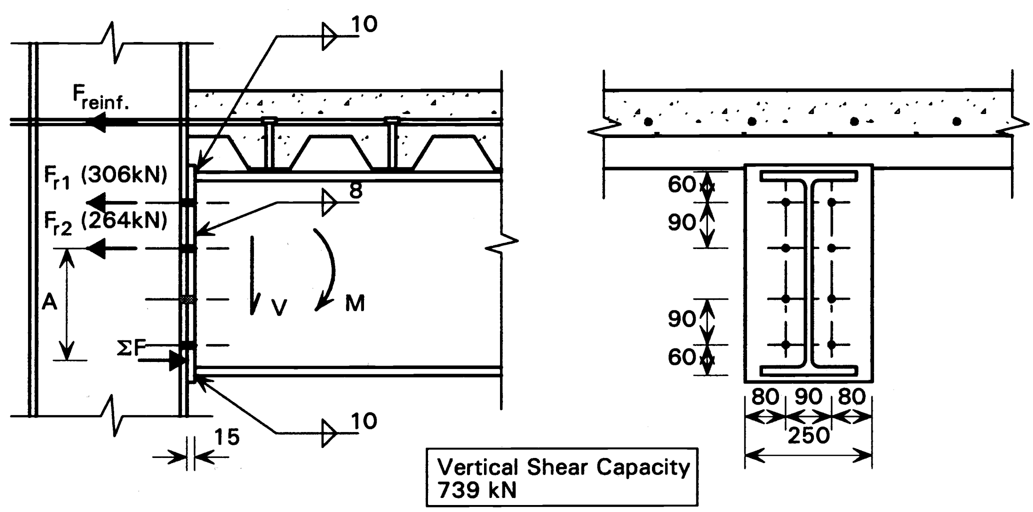
✓ Column satisfactory for bolt row tension values shown for the beam side.

195 Recalculate moment capacity based on reduced bolt row force (195 kN) using dimension 'A' to derive appropriate lever arm - or provide tension stiffener at the appropriate bolt row level.

Compression Zone:

✓ Column capacity exceeds $\Sigma F = F_{reinf} + F_{r1}$

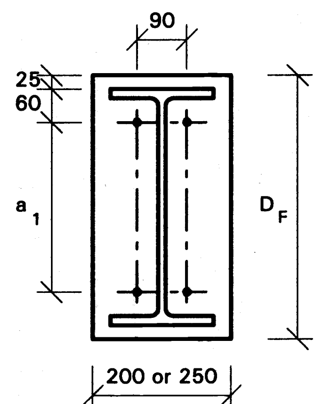
S Provide compression stiffener.



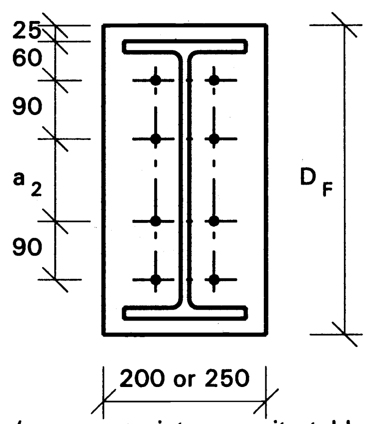
B.3 DETAILING TABLES

DIMENSIONS FOR DETAILING

Beam serial size	Dimension a_1 mm	Dimension a_2 mm	End plate overall depth D_F mm
533x210x122	425	245	600
109	420	240	
101	415	235	
92	415	235	
82	410	230	
457x191x98	350	170	520
89	345	165	
82	340	160	
74	340	160	
67	335	155	
457x152x82	345	165	520
74	340	160	
67	340	160	
60	335	155	
52	330	150	
406x178x74	295	115	470
67	290	110	
60	285	105	
54	285	105	
406x140x46	280	100	450
39	275	95	
356x171x67	245	–	420
57	240	–	
51	235	–	
45	230	–	
356x127x39	235	–	410
33	230	–	
305x165x54	190	–	360
46	185	–	
40	185	–	
305x127x48	190	–	360
42	185	–	
37	185	–	
305x102x33	195	–	370
28	190	–	
25	185	–	
254x146x43	140	–	310
37	135	–	
31	135	–	
254x102x28	140	–	310
25	135	–	
22	135	–	



(see appropriate capacity table)



(see appropriate capacity table)

See capacity table diagram for plate thickness and other dimensions appropriate to the moment capacities.
All plates to be S275.