Joints in Steel Construction
Composite Connections

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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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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 1.4\(\sigma_y\) 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 1.2\(\sigma_y\). 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 ‘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\(^1,2,3,4\).

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
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- 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 anchoring or 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.
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**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).

**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).
Introduction

- 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
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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.

Table 1.1 General methods of ULS design

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

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.

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.
Composite Connections

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* 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
- **\gamma_{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\(^\text{(7,10)}\). 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 even though 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](image-url)
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(15)).

Tests and models(18) 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](image)

**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 1.4\(p_L\). The factor of 1.4 allows for strain hardening and some dispersion into the web at the root of the section\(^{(5)}\). 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 \(p_L\). 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\(^{11}\) 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, $\alpha = 0.85$)
Figure 3.1 shows schematically how the free bending moment \( \left( \frac{w^2}{8} \right) \) 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 \( \alpha \), 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 \( \alpha \) 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 \( \frac{w^2}{8} \) 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 = \( \frac{w^2}{8} = 570 \) 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.

\begin{center}
\begin{tabular}{|c|c|c|}
\hline
Beam span & 10 m \\
Dead load & 4 kN/m\(^2 \) \\
Imposed load & 6 kN/m\(^2 \) \\
Loaded width & 3 m \\
\hline
\end{tabular}
\end{center}
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<thead>
<tr>
<th>BEAM Serial Size</th>
<th>S275</th>
<th>S355</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.85 $M_p$</td>
<td>BEAM + CONNECTION</td>
</tr>
<tr>
<td></td>
<td>$M_{TMIN}$</td>
<td>$M_{TMAX}$</td>
</tr>
<tr>
<td>533x210x122</td>
<td>1200</td>
<td>1563</td>
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<td>980</td>
<td>1338</td>
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<tr>
<td>51</td>
<td>920</td>
<td>1277</td>
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<td>82</td>
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<td>457x191x398</td>
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</tr>
<tr>
<td>22</td>
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<td>144</td>
</tr>
</tbody>
</table>

*M_p* values are based on typical composite beam details, assuming a slab depth of 120 mm, and full shear connection. $M_{TMIN}$ 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).
Use Table 3.1 to estimate beam and connection requirements

Check that for UDL, UD ≤ 25, 2PL, UD ≤ 20, 1PL, UD ≤ 25

Are bare steel beam and connections adequate at construction stage?

Check construction loads on bare steel beam and connections

Calculate beam $M_0$ (initially assuming full interaction)

Compare free moment and $0.85M_0$ to determine minimum connection moment resistance required

Detail suitable 'plastic' connection (bolts, reinforcement and shear connectors)

Choose a suitable standard connection using design tables

Can required connection strength be achieved?

Does column require stiffening?

Is increase in column size economic alternative?

Detail shear connectors (beam) to achieve required interaction

Is connector spacing acceptable?

Calculate dead load deflection of steel beam $\delta_{dead}$

Calculate imposed load deflection of composite beam $\delta_{imp}$

Calculate total deflection (if appropriate) $\delta_t = \delta_{dead} + \delta_{imp}$

Calculate natural frequency of the beam

Does natural frequency exceed excitation frequency?

Check vibration response using procedure in Ref. 16

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, 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 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, 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.
Composite Connections

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 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
Composite Connections

START

Set trial configuration

STEP 0 (Optional)
Calculate the moment capacity at the construction stage (bare steel)

STEP 1
Calculate the resistances of the reinforcement and bolt rows in tension

STEP 2
Calculate the resistance in the compression zone

STEP 3
Calculate the resistance of the column web panel in shear

STEP 4
Adjust the resistances from STEP 1 to ensure equilibrium. Calculate the moment capacity, $M_c$

Is $M_c >$ Design Moment?

No

Yes

STEP 5
Design for vertical shear forces

STEP 6
Design the stiffeners

STEP 7
Design the welds

END

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
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.:

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

$$P_{r2} = \text{Min. of:}$$

- resistance of row 2 alone
- (resistance of rows 2 + 1) - $P_{r1}$

$$P_{r3} = \text{Min. of:}$$

- 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.
**STEP 1A REINFORCEMENT YIELDING**

**Potential resistance**

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

\[ P_{\text{reinf}} = \frac{f_y A_{\text{reinf}}}{Y_m} \]  

(1.1)

- \( f_y \) = design yield strength of reinforcement
- \( A_{\text{reinf}} \) = area of reinforcement within the effective width of slab for the connection (see Figure 4.6 and Section 5).
- \( Y_m \) = partial safety factor for reinforcement (taken as 1.05)

**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\(^{(20)}\).
STEP 1A REINFORCEMENT YIELDING (continued)

![Graph showing stress-strain relationship with total elongation and minimum elongation at fracture.]

Minimum elongation at fracture:
- \( \geq 5\% \) for BS 500B to DD ENV 10080
- \( \geq 14\% \) according to BS 4449

Figure 4.5 Elongation limits for reinforcement

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

<table>
<thead>
<tr>
<th>Type</th>
<th>Steel</th>
<th>Rebar Elongation Limit</th>
<th>Beam Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>203</td>
</tr>
<tr>
<td>Compact</td>
<td>S275</td>
<td>5%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>S355</td>
<td>5%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10%</td>
<td>500</td>
</tr>
<tr>
<td>Plastic</td>
<td>S275</td>
<td>5%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>S355</td>
<td>5%</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10%</td>
<td>500</td>
</tr>
</tbody>
</table>

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. 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.

![Figure 4.6 Truss model for connection behaviour under unbalanced moment](image)

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

\[
A_L \leq \frac{0.6 b_o d_s f_{cu}}{\mu f_y} \quad (1.2)
\]

where:
- \(b_o\) = width of column
- \(d_s\) = depth of slab above decking
- \(f_{cu}\) = cube strength of concrete
- \(f_y\) = yield strength of the rebar

\(\mu\) is a function of the difference in applied moments, and beam depths, either side of the node:

\[
\mu = 1 - \frac{M_{low}}{M_{high}} \frac{h_1}{h_2} \quad (1.3)
\]

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 \(\mu\) 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:

\[
A_T \geq \frac{0.35 \mu A_L}{\left(\frac{\sigma_T}{\sigma_L} - 0.3\right)} \quad (1.4)
\]
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

<table>
<thead>
<tr>
<th>Column serial size</th>
<th>Allowable area of reinforcement (mm²)</th>
<th>1 row M20</th>
<th>2 rows M20</th>
<th>1 row M24</th>
<th>2 rows M24</th>
</tr>
</thead>
<tbody>
<tr>
<td>356x368x202</td>
<td></td>
<td>2160</td>
<td>1851</td>
<td>1936</td>
<td>1413</td>
</tr>
<tr>
<td>356x368x177</td>
<td></td>
<td>1664</td>
<td>1356</td>
<td>1440</td>
<td>918</td>
</tr>
<tr>
<td>356x368x153</td>
<td></td>
<td>1271</td>
<td>963</td>
<td>1048</td>
<td>525</td>
</tr>
<tr>
<td>356x368x129</td>
<td></td>
<td>906</td>
<td>598</td>
<td>683</td>
<td>160</td>
</tr>
<tr>
<td>305x305x198</td>
<td></td>
<td>2783</td>
<td>2474</td>
<td>2559</td>
<td>2036</td>
</tr>
<tr>
<td>305x305x158</td>
<td></td>
<td>1897</td>
<td>1589</td>
<td>1673</td>
<td>1150</td>
</tr>
<tr>
<td>305x305x137</td>
<td></td>
<td>1477</td>
<td>1169</td>
<td>1253</td>
<td>730</td>
</tr>
<tr>
<td>305x305x118</td>
<td></td>
<td>1105</td>
<td>797</td>
<td>881</td>
<td>358</td>
</tr>
<tr>
<td>305x305x97</td>
<td></td>
<td>788</td>
<td>479</td>
<td>564</td>
<td>41</td>
</tr>
<tr>
<td>254x254x167</td>
<td></td>
<td>2659</td>
<td>2351</td>
<td>2436</td>
<td>1913</td>
</tr>
<tr>
<td>254x254x132</td>
<td></td>
<td>1783</td>
<td>1475</td>
<td>1559</td>
<td>1036</td>
</tr>
<tr>
<td>254x254x107</td>
<td></td>
<td>1223</td>
<td>915</td>
<td>1000</td>
<td>477</td>
</tr>
<tr>
<td>254x254x89</td>
<td></td>
<td>797</td>
<td>488</td>
<td>573</td>
<td>50</td>
</tr>
<tr>
<td>254x254x73</td>
<td></td>
<td>520</td>
<td>212</td>
<td>297</td>
<td>0</td>
</tr>
<tr>
<td>203x203x86</td>
<td></td>
<td>1125</td>
<td>817</td>
<td>902</td>
<td>379</td>
</tr>
<tr>
<td>203x203x71</td>
<td></td>
<td>694</td>
<td>386</td>
<td>470</td>
<td>0</td>
</tr>
<tr>
<td>203x203x60</td>
<td></td>
<td>530</td>
<td>221</td>
<td>306</td>
<td>0</td>
</tr>
<tr>
<td>203x203x52</td>
<td></td>
<td>347</td>
<td>39</td>
<td>123</td>
<td>0</td>
</tr>
<tr>
<td>203x203x46</td>
<td></td>
<td>240</td>
<td>0</td>
<td>16</td>
<td>0</td>
</tr>
</tbody>
</table>

Note:  
4x16 mm bars = 804 mm²  
6x16 mm bars = 1210 mm²  
8x16 mm bars = 1610 mm²  
10x16 mm bars = 2010 mm²  
4x20 mm bars = 1260 mm²  
6x20 mm bars = 1890 mm²  
8x20 mm bars = 2510 mm²  
10x20 mm bars = 3140 mm²

Values are based on \(Y_m = 1.05\) and \(f_{sk} = 460\) N/mm²
### Table 4.2 (b) Maximum areas of reinforcement that can be adopted without column compression stiffening, S355 columns

<table>
<thead>
<tr>
<th>Column serial size</th>
<th>Allowable area of reinforcement (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 row M20</td>
</tr>
<tr>
<td>356x368x202</td>
<td>2926</td>
</tr>
<tr>
<td>356x368x177</td>
<td>2287</td>
</tr>
<tr>
<td>356x368x153</td>
<td>1780</td>
</tr>
<tr>
<td>356x368x129</td>
<td>1324</td>
</tr>
<tr>
<td>305x305x198</td>
<td>3730</td>
</tr>
<tr>
<td>305x305x158</td>
<td>2586</td>
</tr>
<tr>
<td>305x305x137</td>
<td>2045</td>
</tr>
<tr>
<td>305x305x118</td>
<td>1564</td>
</tr>
<tr>
<td>305x305x97</td>
<td>1153</td>
</tr>
<tr>
<td>254x254x167</td>
<td>3570</td>
</tr>
<tr>
<td>254x254x132</td>
<td>2440</td>
</tr>
<tr>
<td>254x254x107</td>
<td>1717</td>
</tr>
<tr>
<td>254x254x89</td>
<td>1180</td>
</tr>
<tr>
<td>254x254x73</td>
<td>810</td>
</tr>
<tr>
<td>203x203x86</td>
<td>1591</td>
</tr>
<tr>
<td>203x203x71</td>
<td>1045</td>
</tr>
<tr>
<td>203x203x60</td>
<td>822</td>
</tr>
<tr>
<td>203x203x52</td>
<td>584</td>
</tr>
<tr>
<td>203x203x46</td>
<td>447</td>
</tr>
</tbody>
</table>

**Note:**
- 4x16 mm bars = 804 mm²
- 6x16 mm bars = 1210 mm²
- 8x16 mm bars = 1610 mm²
- 10x16 mm bars = 2010 mm²
- 4x20 mm bars = 1260 mm²
- 6x20 mm bars = 1890 mm²
- 8x20 mm bars = 2510 mm²
- 10x20 mm bars = 3140 mm²

Values are based on $\gamma_m = 1.05$ and $f_{sk} = 460$ N/mm²
**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**

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

**Mode 2 Bolt failure with flange/end plate yielding**

$$P_r = \frac{2M_p + n(\Sigma P'_t)}{m + n} \quad (1.6)$$

**Mode 3 Bolt failure**

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

where:

- $P_r = \text{potential resistance of the bolt row, or bolt group}$
- $P'_t = \text{enhanced bolt tension capacity where prying is taken into account (see Table 4.3)}$
- $\Sigma P'_t = \text{total tension capacity for all the bolts in the group}$
- $M_p = \text{plastic moment capacity of the equivalent T-stub representing the column flange or end plate}$
- $L_{eff} = \text{effective length of yield line in equivalent T-stub (see Tables 4.4, 4.5 and 4.6)}$
- $t = \text{column flange or end plate thickness}$
- $p_y = \text{design strength of column/end plate}$
- $m = \text{distance from bolt centre to 20% distance into column root or end plate weld (see Figure 4.8)}$
- $n = \text{effective edge distance (see Figure 4.8)}$

$M_p = L_{eff} t^2 p_y \quad (1.8)$
STEP 1B (continued)

Table 4.3  Tensile capacity of a single 8.8 bolt

<table>
<thead>
<tr>
<th>Bolt Size</th>
<th>BS 5950: Part 1: ( P_t )</th>
<th>Enhanced value: ( P'_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(450 N/mm(^2))</td>
<td>(560 N/mm(^2))</td>
</tr>
<tr>
<td>M20</td>
<td>110kN</td>
<td>137kN</td>
</tr>
<tr>
<td>M24</td>
<td>159kN</td>
<td>198kN</td>
</tr>
<tr>
<td>M30</td>
<td>252kN</td>
<td>314kN</td>
</tr>
</tbody>
</table>

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)

Back-up 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.

Figure 4.7  Influence of stiffeners
Table 4.4 $L_{\text{eff}}$ for equivalent T-stubs for bolt row acting alone

| Pattern (i) | Circular yielding | $L_{\text{eff}} = 2\pi m$ |
| Pattern (ii) | Side yielding | $L_{\text{eff}} = 4m + 1.25e$ |
| Pattern (iii) | Side yielding near beam flange or a stiffener | $L_{\text{eff}} = \alpha m_1$ |
| Pattern (iv) | Side yielding between two stiffeners | $L_{\text{eff}} = \alpha m_1 + \alpha' m_1 - (4m_1 + 1.25e)$ ($\alpha$ is calculated using $m_{2u}$, $\alpha'$ is calculated using $m_{2l}$) |
| Pattern (v) | Corner yielding | $L_{\text{eff}} = 2m + 0.625e + e_x$ |
| Pattern (vi) | Corner yielding near a stiffener | $L_{\text{eff}} = \alpha m_1 - (2m_1 + 0.625e) + e_x$ |

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 $\alpha$ is determined from the chart in Figure 4.9.
- $L_{\text{eff}}$ is the length of the equivalent T-stub, not the length of the pattern shown.
Table 4.5 \( L_{\text{eff}} \) to be considered for a bolt row acting alone (expressions to be used from Table 4.4)

<table>
<thead>
<tr>
<th>Bolt row not influenced by a stiffener or a free end</th>
<th>Bolt row between stiffeners</th>
</tr>
</thead>
<tbody>
<tr>
<td>Use: ( \text{Min}{\text{ii}, \text{i}} )</td>
<td>Use: ( \text{Min}{\text{Max}{\text{iv}, \text{iii} (m_2), \text{iii}(m_2), \text{ii}, \text{i}}} )</td>
</tr>
<tr>
<td></td>
<td>( (m_2, m_2, m_2, m_2) ) as shown in pattern iv</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bolt row next to a stiffener</th>
<th>Bolt row next to a free end</th>
</tr>
</thead>
<tbody>
<tr>
<td>Use: ( \text{Min}{\text{Max}{\text{ii}, \text{iii}}, \text{i}} )</td>
<td>Use: ( \text{Min}{\text{ii}, \text{i}} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bolt row below the beam flange</th>
<th>Bolt row between free end and stiffeners</th>
</tr>
</thead>
<tbody>
<tr>
<td>If ( g &gt; 0.7B_b ) or ( T_b &lt; 0.8t )</td>
<td>Use: ( \text{Min}{\text{Max}{\text{ii} \div 2, \text{iii}}, \text{i}} )</td>
</tr>
<tr>
<td>Otherwise</td>
<td>Use: ( \text{Min}{\text{Max}{\text{ii}, \text{iii}}, \text{i}} )</td>
</tr>
<tr>
<td>(this is an interim rule)</td>
<td>( (m_2, m_2, m_2, m_2) )</td>
</tr>
</tbody>
</table>

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.
- \( \text{Min}\{\text{Max}\{\text{ii}, \text{iii}\}, \text{i}\} \) 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)

<table>
<thead>
<tr>
<th>Description</th>
<th>Expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top or bottom row of a group along a clear length</td>
<td>Use: $\frac{ii}{2} + \frac{p}{2}$</td>
</tr>
<tr>
<td>Intermediate row of a group</td>
<td>Use: $p$</td>
</tr>
<tr>
<td>Bolt row of a group below the beam flange</td>
<td>If $g &gt; 0.7B_b$ or $T_b &lt; 0.8t_p$ Use: $\frac{ii}{2} + \frac{p}{2}$ if $g &gt; 0.7B_b$ or $T_b &lt; 0.8t_p$</td>
</tr>
<tr>
<td></td>
<td>otherwise Use: $\max \left( \frac{ii}{2}, \frac{ii}{2} + \frac{p}{2} \right)$</td>
</tr>
</tbody>
</table>

Typical Examples

- **Group of three rows in a clear length**
  
  $L_{eff} = \text{pattern (ii)} + 2p$

  $L_{eff}$ for group = $4m + 1.25e + 2p$

- **Group of three rows in a flush end plate**

  $L_{eff} = \max \left( \frac{ii}{2}, \frac{ii}{2} + 2p \right)$ or $\frac{ii}{2} + 2p + \frac{0.5am_i}{2}$

  $L_{eff}$ for group = $\max \left( 4m + 1.25e + 2p \right)$ or $0.5am_i + 2m + 0.625 + 2p$ 

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.
For the end plate:

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

\[
e = \frac{b_p - g}{2}\]

For the column flange:

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

\[
e = \frac{B - g}{2}\]

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.
Figure 4.9  Values of $\alpha$ for bolt rows adjacent to a stiffener or beam flange

Note: Mathematical expressions for $\alpha$ are given in Reference 4

Minimum value of $\alpha = 4.45$

Maximum value of $\alpha = 2\pi$

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

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

\[ P_t = L_t \times t_w \times \rho_y \]  

(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
- \( \rho_y \): 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, \( L_t \).

Potential failure line in column web

Potential failure line in beam web

Rows 1 + 2: Column side

Rows 2: Beam side

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

<table>
<thead>
<tr>
<th>Row</th>
<th>Column Side</th>
<th>Beam Side</th>
<th>Step 1D Modify bolt forces</th>
<th>Potential Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Step 1B Flange Bending</td>
<td>Step 1C Web Tension</td>
<td>Step 1B Plate Bending</td>
<td>Step 1C Web Tension</td>
</tr>
<tr>
<td></td>
<td>Reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Resistance of Row 1**

| Resistance of Row 1 | | |
|---------------------|---|
| 2 | 3 | 4 | 5 | 6 |


**Resistance of Row 2 alone**

| Resistance of Row 2 alone | | |
|---------------------------|---|
| 8 | 9 | 10 | 11 | |

**Resistance of rows (1 + 2) combined:**

| Resistance of rows (1 + 2) combined | | |
|--------------------------------------|---|
| 12 | 13 | 14 | 15 | |

<table>
<thead>
<tr>
<th>Deduct box 6:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>17</td>
</tr>
</tbody>
</table>

**Resistance of Row 3 alone**

| Resistance of Row 3 alone | | |
|---------------------------|---|
| 22 | 23 | 24 | 25 | |

**Resistance of combined rows (2 + 3)**

| Resistance of combined rows (2 + 3) | | |
|-------------------------------------|---|
| 26 | 27 | 28 | 29 | |

<table>
<thead>
<tr>
<th>Deduct box 20:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>31</td>
</tr>
</tbody>
</table>

**Resistance of combined rows (1 + 2 + 3):**

| Resistance of combined rows (1 + 2 + 3) | | |
|----------------------------------------|---|
| 34 | 35 | 36 | 37 | |

**Deduct sum of boxes 6 & 20:**

<table>
<thead>
<tr>
<th>Deduct sum of boxes 6 &amp; 20</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>39</td>
</tr>
</tbody>
</table>

See the worked example using the worksheet in Appendix A
**Beam-to-Column Connections**

**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](image)

\[ P_c = (b_1 + n_2) \times t_c \times \rho_c \]  \hspace{1cm} (1.14)

where:

- \( b_1 \) = stiff bearing length as above
- \( n_2 \) = 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
- \( \rho_c \) = compressive strength of the column web from BS 5950: Part 1 Table 27(c) with \( \lambda = 2.5d/t_c \) for web 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

**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](image)

\[ P_c = (b_1 + n_1) \times t_c \times \rho_c \]  \hspace{1cm} (1.15)

where:

- \( b_1 \) = stiff bearing length as above
- \( n_1 \) = length obtained by a 1:2.5 dispersion through the column flange and root radius
- \( t_c \) = column flange thickness
- \( \rho_c \) = design strength of the column
- \( \rho_p \) = end plate thickness
- \( D_c \) = column flange thickness
- \( r \) = column root radius
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:

\[ P_c = 1.4 \times \rho_{yb} \times T_b \times B_b \]  \hspace{1cm} (1.16)

where:

- \( \rho_{yb} \) = design strength of the beam
- \( T_b \) = the beam flange thickness
- \( B_b \) = the beam flange breadth (with due consideration of any notching)

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 1.4\( \rho_{yb} \) can be taken when the flange area is assumed to act alone(41). 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 \( \perp \) shaped compression zone should be considered, extending some distance up the web (see Figure 4.15).

Note that:

- The stress in this \( \perp \) section must be limited to 1.2\( \rho_{y} \), 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 \( \perp \) section, and the lever arms to the bolts and reinforcement are reduced accordingly.

Because compression in the beam extends into the web, and a limiting stress of 1.2\( \rho_{y} \) 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 19(\( t_c \) adjacent to both the compression flange and the neutral axis (where \( t_c \) is the web thickness and \( c = \sqrt{\rho_{y}/275} \)).

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.
### 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:

$$P_v = 0.6 \times p_{vc} \times t_c \times D_c$$

where

- $p_{vc}$ = design strength of the column
- $t_c$ = column web thickness
- $D_c$ = column section depth

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 resultant panel shear from connections to both column flanges must be taken into account.

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

![Figure 4.16 Web panel subject to shear force](image-url)
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:

\[
F_{\text{reinf}} + \sum F_i + N = F_c \quad (1.18)
\]

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

and \( F_c \) is the smallest of the following:

\[
P_{\text{reinf}} + \sum P_i + N \quad (1.19)
\]

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: \( F_{\text{reinf}} \leq P_{\text{reinf}} \)

For each bolt row: \( F_i \leq P_i \)

where:

\[
\begin{align*}
P_{\text{reinf}} &= \text{potential resistance of reinforcement} \\
P_i &= \text{potential resistance of bolt row } i \\
F_{\text{reinf}} &= \text{final force in reinforcement} \\
F_i &= \text{final force in bolt row } i
\end{align*}
\]

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:

\[
M_j = F_{\text{reinf}} \times h_{\text{reinf}} + \sum (F_{ri} \times h_i) \quad (1.20)
\]

where:
- \(h_{\text{reinf}}\) = 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:

\[
M_m = M_j + (N \times h_N) \quad (1.21)
\]

where:
- \(M_j\) = basic connection moment capacity
- \(N\) = axial force
- \(h_N\) = distance of axial force from centre of compression
Composite Connections

**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:

\[
V \leq (n_s \times P_{ss}) + (n_t \times P_{ts}) \quad (1.22)
\]

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
  - \(d_t p_b\) for bolt bearing on the end plate, or
  - \(d_T p_b\) 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
  - \(d_t p_b\) for bolt bearing on the end plate, or
  - \(d_T p_b\) 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

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

**Table 4.7 Shear capacities of single 8.8 bolts**

<table>
<thead>
<tr>
<th>Bolt size</th>
<th>Bolts in shear only kN</th>
<th>Bolts in shear &amp; tension kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>M20</td>
<td>91.9</td>
<td>36.8</td>
</tr>
<tr>
<td>M24</td>
<td>132</td>
<td>53</td>
</tr>
<tr>
<td>M30</td>
<td>210</td>
<td>84.2</td>
</tr>
</tbody>
</table>

*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:

\[
V_1 + V_2 \leq 2P_v \quad (1.23)
\]

\(V_1\) = shear applied from beam 1
STEP 5 DESIGN FOR VERTICAL SHEAR FORCES (continued)

\[ V_2 = \text{shear applied from beam 2} \]

\[ P_v = \text{local shear capacity of column web smaller of } 0.6p_Av \text{ and } 0.5U_sA_{vnet} \]

\[ A_v = [p + (n_t + n_s2 - 2)p/2 + e_1t_c ] \quad (1.24) \]

\[ p = \text{bolt pitch} \]

\[ n_s2 = \text{number of shear bolts on side 2} \]

\[ d = \text{bolt diameter} \]

\[ t_c = \text{column web thickness} \]

\[ A_{vnet} = A_v - (n_s2 + n_t)D_h t_c/2 \quad (1.25) \]

\[ D_h = \text{hole diameter} \]

\[ U_s = \text{ultimate tensile strength of column web} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

(iv) For web bearing:

\[ V_1/(n_{s1} + n_t) + V_2/(n_{s2} + n_t) \leq dt_p p_b \quad (1.30) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (web)} \]

\[ V_2 = \text{shear applied from beam 2} \]

\[ V_1 = \text{local shear capacity of column web smaller of } 0.6p_Av \text{ and } 0.5U_sA_{vnet} \]

\[ A_v = [p + (n_t + n_s2 - 2)p/2 + e_1t_c ] \quad (1.24) \]

\[ p = \text{bolt pitch} \]

\[ n_s2 = \text{number of shear bolts on side 2} \]

\[ d = \text{bolt diameter} \]

\[ t_c = \text{column web thickness} \]

\[ A_{vnet} = A_v - (n_s2 + n_t)D_h t_c/2 \quad (1.25) \]

\[ D_h = \text{hole diameter} \]

\[ U_s = \text{ultimate tensile strength of column web} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

(iv) For web bearing:

\[ V_1/(n_{s1} + n_t) + V_2/(n_{s2} + n_t) \leq dt_p p_b \quad (1.30) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (web)} \]

\[ V_2 = \text{shear applied from beam 2} \]

\[ V_1 = \text{local shear capacity of column web smaller of } 0.6p_Av \text{ and } 0.5U_sA_{vnet} \]

\[ A_v = [p + (n_t + n_s2 - 2)p/2 + e_1t_c ] \quad (1.24) \]

\[ p = \text{bolt pitch} \]

\[ n_s2 = \text{number of shear bolts on side 2} \]

\[ d = \text{bolt diameter} \]

\[ t_c = \text{column web thickness} \]

\[ A_{vnet} = A_v - (n_s2 + n_t)D_h t_c/2 \quad (1.25) \]

\[ D_h = \text{hole diameter} \]

\[ U_s = \text{ultimate tensile strength of column web} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

(iv) For web bearing:

\[ V_1/(n_{s1} + n_t) + V_2/(n_{s2} + n_t) \leq dt_p p_b \quad (1.30) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (web)} \]

\[ V_2 = \text{shear applied from beam 2} \]

\[ V_1 = \text{local shear capacity of column web smaller of } 0.6p_Av \text{ and } 0.5U_sA_{vnet} \]

\[ A_v = [p + (n_t + n_s2 - 2)p/2 + e_1t_c ] \quad (1.24) \]

\[ p = \text{bolt pitch} \]

\[ n_s2 = \text{number of shear bolts on side 2} \]

\[ d = \text{bolt diameter} \]

\[ t_c = \text{column web thickness} \]

\[ A_{vnet} = A_v - (n_s2 + n_t)D_h t_c/2 \quad (1.25) \]

\[ D_h = \text{hole diameter} \]

\[ U_s = \text{ultimate tensile strength of column web} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

\[ V_2 = \text{shear applied from beam 2} \]

\[ V_1 = \text{local shear capacity of column web smaller of } 0.6p_Av \text{ and } 0.5U_sA_{vnet} \]

\[ A_v = [p + (n_t + n_s2 - 2)p/2 + e_1t_c ] \quad (1.24) \]

\[ p = \text{bolt pitch} \]

\[ n_s2 = \text{number of shear bolts on side 2} \]

\[ d = \text{bolt diameter} \]

\[ t_c = \text{column web thickness} \]

\[ A_{vnet} = A_v - (n_s2 + n_t)D_h t_c/2 \quad (1.25) \]

\[ D_h = \text{hole diameter} \]

\[ U_s = \text{ultimate tensile strength of column web} \]

(iii) For end plate bearing:

\[ V_1 \leq (n_{s1} + n_t)dt_{p1}p_b \quad (1.28) \]

and

\[ V_2 \leq (n_{s2} + n_t)dt_{p2}p_b \quad (1.29) \]

where:

\[ p_b \text{ is the minimum of } p_{bb} \text{ (bolt)} \text{ or } p_{bs} \text{ (plate)} \]

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*

*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](image)

Effective Outstand of Compression Stiffeners

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

$$b_{sg} = \text{stiffener outstand (see Figure 4.20)}$$

$$t_s = \text{thickness of stiffener}$$

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.)

$$P_c \text{ buckling} = (A_w + A_{sg}) \times p_c \quad (1.31)$$

$$P_c \text{ crushing} = [A_{sn} \times p_y] + [(b_1 + n_2) \times t_c \times p_y] \quad (1.32)$$

and:

$$P_c \text{ bearing} = \frac{A_{sn} \times p_y}{0.8} \quad (1.33)$$

where:

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

$A_{sg}$ = gross area of stiffeners

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

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

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

$p_c$ = compressive strength of stiffeners from BS 5950: Part 1 Table 27(c) with $\lambda = 0.7L/t_y^*$
## STEP 6A DESIGN OF COLUMN COMPRESSION STIFFENERS (continued)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L$</td>
<td>length of stiffener = $D_o - 2T_c$</td>
</tr>
<tr>
<td>$r_y$</td>
<td>radius of gyration of effective area (as shown in Section A-A, Figure 4.20)</td>
</tr>
<tr>
<td>$p_y$</td>
<td>lesser of the design strength of stiffener or column</td>
</tr>
<tr>
<td>$p_{vs}$</td>
<td>design strength of stiffener</td>
</tr>
<tr>
<td>$(b_1 + n_2)$</td>
<td>effective bearing length along web (see Step 2A)</td>
</tr>
</tbody>
</table>

**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.

**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.
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

\[ P_t = \frac{4M_p + 2M_{bp}}{m} \]  

(1.36)

Mode 2 Bolt failure with flange yielding

\[ P_t = \frac{2M_p + n\sum P'_t}{m + n} \]  

(1.37)

Mode 3 Bolt failure

\[ P_t = \sum P'_t \]  

(1.38)

where:

\[ M_{bp} = \frac{L_{eff} \times t_{bp}^2 \times \rho_y}{4} \]  

(1.39)

\[ t_{bp} = \] thickness of the backing plate

\[ \rho_y = \] 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 where flanges are particularly thin. The plates are generally supplied loose or tack-welded in place and their effect is to prevent or increase the resistance to a Mode 1 bending failure. Neither Mode 2 nor Mode 3 is affected.
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:

$$A_{sn} = \frac{(F_{ri} + F_{rj})}{\rho_y} = (L_t \times t_c)$$

(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

$\rho_y =$ 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)

$$t_c = \text{web thickness}$$

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

Flange Bending
The force carried by the stiffeners is assumed to be inversely proportional to their distance from the bolts.
Beam-to-Column Connections

STEP 6C DESIGN OF TENSION STIFFENERS (continued)

Basic requirement:

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

where:

$$A_{sn} = \text{net area of both stiffeners}$$

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

$$\rho_v = \text{the design strength of stiffener or column (the lesser of the two).}$$

$m_1$, $m_{2L}$, $m_{2U}$, $F_n$ and $F_{ij}$ are defined in Figure 4.24

Row i, Tension = $F_{ni}$ kN

Row j, Tension = $F_{nj}$ kN

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

<table>
<thead>
<tr>
<th>General</th>
<th>Column Web Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>A supplementary web plate (SWP) may be provided to increase the capacity of the column web. Its effect (according to EC3) is to:</td>
<td>For the purpose of the column web tension calculation (Step 1C), the effective web thickness, ( t_{\text{eff}} ), should be taken as:</td>
</tr>
</tbody>
</table>
| - Increase web tension resistance by: 50% with a plate on one side, or 100% with plates on both sides | - For an SWP on one side only, \( t_{\text{eff}} = 1.5t_c \)  
- For SWPs on both sides, \( t_{\text{eff}} = 2t_c \)  
  \( t_c \) is the column web thickness. |
| - Increase web crushing resistance by: 50% with a plate on one side, or 100% with plates on both sides | - For an SWP on one side only, \( t_{\text{eff}} = 1.5t_c \)  
- For SWPs on both sides, \( t_{\text{eff}} = 2t_c \)  
  \( t_c \) is the column web thickness. |
| - Increase web panel shear resistance by: about 75% (see expression for \( P_v \)). | Column Web Crushing and Buckling  
For the purpose of column web crushing and buckling calculations (Step 2A), the effective web thickness, \( t_{\text{eff}} \), should be taken as: |
| Note that in the case of panel shear, plates on both sides provide no additional increase over a plate on one side. | - For an SWP on one side only, \( t_{\text{eff}} = 1.5t_c \)  
- For SWPs on both sides, \( t_{\text{eff}} = 2t_c \)  
  \( t_c \) is the column web thickness. |

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 37\( t_s \) (for S275 steel) or 33\( t_s \) (S355).

- **Breadth**, \( b_s \) that satisfies:
  \[ b_s \geq d - 2t_s \]  
  \( (b_s = d \) for a "fill in" weld)  
  \[ b_s \geq d - 2t_s \]  

- **Length**, \( L_s \geq g + D_b + D_c/2 + D_r + e \)  
  \[ L_s \geq g + D_b + D_c/2 + D_r + e \]  
  \[ L_s \geq g + D_b + D_c/2 + D_r + e \]  

\[ L_s \geq g + D_b + D_c/2 + D_r + e \]  

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)  

\[ L_s \geq g + D_b + D_c/2 + D_r + e \]  

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

\[ P_v = 0.6 \times p_y \times A_v \]  

\[ P_v = 0.6 \times p_y \times A_v \]  

where:
- \( p_y \) = design strength of the column  
- \( A_v \) = shear area of the column web and SWP combined  
  \[ A_v = t_c \times (D_c + b_s) \]  

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)

Plug welds dia \( \geq t_s \) spaced \( \leq 37 \ t_s \) when \( b_s > 37 \ t_s \) for grade S275

Continuous fillet weld. Leg length equal to plate thickness

'Fill in' butt weld where increased web tension resistance is needed

Figure 4.25 Dimensions and welds
Composite Connections

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,
   \[ B \times T \times \sigma_y \quad (1.46) \]

(b) The total tension force in the top two bolt rows (see Figure 4.26)
   \[ F_{t1} + F_{t2} \quad (1.47) \]

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.

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/mm² for S275 and 255 N/mm² for S355).
- The shear and tension stresses on the fusion lines should not exceed 0.7\(\sigma_y\) and 1.0\(\sigma_y\) 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."
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:

   $$ P_{sw} = 2 \times a \times \rho_w \times L_{ws} \quad (1.48) $$

   where:
   - $a = \text{fillet weld throat thickness} \times 0.7s_w$
   - $\rho_w = \text{design strength of fillet weld (BS 5950: Part 1 Table 36)}$
   - $L_{ws} = \text{length of shear zone welds} = D_b - 2(T_b + r_b) - L_{wt} \quad (1.49)$

   **Figure 4.27** A partial penetration butt weld with superimposed fillets

   **Figure 4.28** Force distribution in welds

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

\[ P_{\text{reinf}} = \frac{f_y A_{\text{reinf}}}{\gamma_m} \]  \hspace{1cm} (2.1)

- \( f_y \) = design yield strength of reinforcement
- \( A_{\text{reinf}} \) = area of reinforcement within the allowable width of slab (see Figure 5.2).
- \( \gamma_m \) = 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:

\[ P_c = 1.4 \times \rho_{yb} \times T_b \times B_b \]  \hspace{1cm} (2.2)

where:

- \( \rho_{yb} \) = 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 1.4\( \rho_{yb} \) 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 \( \perp \) shaped compression zone should be considered, extending some distance up the web (see Figure 4.31).

Note that:

- The stress in this \( \perp \) section must be limited to 1.2\( \rho_y \), 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 \( \perp \) 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 1.2\( \rho_y \) 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\( e \) adjacent to both the compression flange and the neutral axis (where \( t_e \) is the web thickness and \( e = \sqrt{\rho_y / 275} \)).
### 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:

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

where $N$ is the axial load in the beam (positive for compression) and $F_c$ is the smaller of the following:

- $P_{\text{reinf}} + N$ (beam crushing)
- $P_c$ (beam crushing)

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

where:

- $F_{\text{reinf}}$ = 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 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:

$$ M_m = M_j + (N \times h_N) \quad (2.5) $$

where:

- $M_m$ = basic connection moment capacity
- $M_j$ = basic 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 curtailling 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 \(e_L\)).

\[e_L = \text{a function of the column width, rather than the beam span.}\]

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\(^{(11)}\).

\[\text{Keeping the longitudinal bars close to the top of the decking minimises the strain they must undergo to achieve a given rotation.}\]

5. The first shear connector should be at least 100 mm from the face of the column.

\[\text{This limitation ensures that reinforcing bars are strained over a substantial length, so that sufficient rotation can take place.}\]

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.
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. 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 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 curtailting 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. 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.

3. The first shear connectors on the supported beams should be at least 200 mm from the centre line of the support.
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  
    Part 1.1 General rules and rules for buildings  
    BSI, 1994

11. BRITISH STANDARDS INSTITUTION  
    BS 8110: The structural use of concrete  
    BSI, 1997

12. AHMED, B., and NETHERCOT, D. A.  
    Effect of high shear on the moment capacity of composite cruciform end plate connections  

    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
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

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
2. Tension zone
3. Compression zone
4. Column panel shear zone
5. Calculation of moment capacity
6. Calculation of vertical shear capacity
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

Potential resistance

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

\[
P_{\text{reinf}} = \frac{f_y \times A_{\text{reinf}}}{Y_m} = \frac{460 \times 1210 \times 10^{-3}}{1.05} = 530.1 \text{ kN}
\]

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 mm² < 1210 mm²  OK

Check maximum area of reinforcement

To prevent concrete crushing:

\[
A_l \times \frac{0.6 \times b_c \times d_s}{\mu} \times \frac{f_{\text{cu}}}{f_y} = \frac{0.6 \times 204.3 \times (120 - 50)}{460} \times 30
\]
For this example, it is assumed that a connection of equal strength will be adopted on the opposing column flange, \( \mu = 0 \), so \( A_L \leq \infty \). So \( A_{\text{reinf}} = 1210 \text{ mm}^2 \) 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):

\[
m = \frac{g - t_c}{2} - 0.8 \times r = \left( \frac{90}{2} - \frac{7.9}{2} - 0.8 \times 10.2 \right) = 32.9 \text{ mm}
\]

\[
e = \frac{B - g}{2} = \left( \frac{204.3 - 90}{2} \right) = 57.2 \text{ mm}
\]

\[
n = \text{minimum of } e, 1.25m \text{ or } e \text{ (beam side, see below)}
\]

\[
= 57.2 \text{ or } 1.25 \times 32.9 \text{ or } 55 = 41.1 \text{ mm}
\]

#### Column flange bending

From Table 4.5, \( L_{\text{eff}} \) is given by Min \{ii,i\}

From Table 4.4, Pattern (i):

\[
2nm = 2\pi \times 32.9 = 206.7 \text{ mm}
\]

Table 4.4, Pattern (ii):

\[
4m + 1.25e = 4 \times 32.9 + 1.25 \times 57.2
\]

Hence \( L_{\text{eff}} = 203.1 \text{ mm} \)

**\( M_p \) for the column flange:**

\[
M_p = \frac{L_{\text{eff}} \times T_c^2 \times p_y}{4} = \frac{203.1 \times 12.5^2 \times 275 \times 10^{-3}}{4} = 2181.7 \text{ kNm}
\]

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.**

\[
P_r = \frac{2M_p + n \times \Sigma P_i}{m + n} = \frac{2 \times 2181.7 + 41.1 \times 2 \times 137}{32.9 + 41.1} = 211.1 \text{ kN}
\]
Mode 3. \[ P_r = \Sigma P_r' = 2 \times 137 = 274 \text{kN} \]

Hence \( P_r = 211.1 \text{kN} \)

Column web tension

\[ L_t = 45 \times 1.73 \times 2 = 155.7 \text{mm} \]

\[ P_t = L_t \times t_w \times p_r = 155.7 \times 7.9 \times 275 \times 10^3 = 338 \text{kN} \]

Beam side geometry:

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

(assume 8FW)

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

\[ n = \text{minimum of } e, 1.25m \text{ or } e \text{ (column side, see above)} \]

\[ = 57.2 \text{ or } 1.25 \times 34.8 \text{ or } 55 = 43.5 \text{ mm} \]

End plate bending

From tables, \( L_{ef} \) is given by:

\[ \text{Min } \{ \text{Max (ii, iii, i) since } g \times 0.7 B_p \text{ and } T_b \times 0.8 t_p \} \]

From Table 4.4 Pattern (ii):

\[ 4m + 1.25e = 4 \times 34.8 + 1.25 \times 55 = 208.0 \text{ mm} \]

Table 4.4 Pattern (iii):

\[ L_{ef} = \alpha m, \text{ where } \alpha \text{ is obtained from Figure 4.9.} \]

Using:

\[ m_1 = m = 34.8 \text{ mm} \]

\[ m_2 = p_t - T_c - 0.8 \times s_{wf} = 60 - 10.9 - (0.8 \times 10) = 41.1 \text{ mm} \]
\[ \lambda_1 = \frac{m_1}{m_1 + e} = \frac{34.8}{34.8 + 55} = 0.39 \]
\[ \lambda_2 = \frac{m_2}{m_1 + e} = \frac{41.1}{34.8 + 55} = 0.46 \]

So from Figure 4.9 \( \alpha = 2\pi \)

\[ \alpha m_1 = 218.7 \text{ mm} \]

\. Max (ii, iii) \( = 218.7 \text{ mm} \)

Pattern (i):

\[ 2\pi m = 218.7 \text{ mm} \]

Hence \( L_{\text{eff}} = 218.7 \text{ mm} \)

So, \( M = \frac{L_{\text{eff}} \times t^2 \times p_1}{4} = \frac{218.7 \times 12^2 \times 275 \times 10^{-3}}{4} \]

\[ = 2165.1 \text{ kNm} \]

Critical failure mode is minimum of:

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

Mode 2. \( P = \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} \]

Mode 3. \( P = \sum P_t = 2 \times 137 = 274 \text{ kN} \)

Beam web tension

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

3.1 **Column Web Crushing**

\[
b_1 = T_b + 2 s_{wr} + 2 t_p = 10.9 + 2 \times 10 + 2 \times 12 = 54.9 \text{ mm}
\]

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

\[
P_c = (b_1 + n) \times t_c \times p_y = (54.9 + 113.5) \times 7.9 \times 275 \times 10^3
\]

\[
= 365.8 \text{ kN}
\]

3.2 **Column Web Buckling**

\[
n_1 = 206.2 \text{ mm}
\]

\[
p_c \text{ is obtained from Table 27(c) of BS 5950 Part 1 using:}
\]

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

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

\[
P_c = 451.7 \text{ kN}
\]

Therefore, column compression resistance \( = 366 \text{ kN} \)

3.3 **Beam Flange Crushing**

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**

For this balanced situation, column panel shear is zero.

5. **CALCULATION OF MOMENT CAPACITY**

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:

\[
P_{\text{reinf}} + P_{rt} = 530 + 208^* = 738 \text{ kN}
\]

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

The potential compression resistance of the beam flange alone...
\[ P_c = 746 \text{ kN} \]

\[ \therefore \text{None of the beam web will be subject to compression.} \]

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

Beam flange thickness  
\[ T_b = 10.9 \text{ mm} \]

Therefore, try 15 mm thick stiffeners

Let width of each stiffener, \( b_{sg} \)
\[ = 95 \text{ mm} \text{ (}= 6.3t) \]

with a corner snipe of 12 mm

Buckling resistance of stiffener + column

\[ P_{c, \text{buckling}} = (A_w + A_{sg}) \times P_c \]

where  
\[ A_w = 40 \times t_c \times t_c = (40 \times 7.9) \times 7.9 = 2496 \text{ mm}^2 \]

\[ A_{sg} = 2 \times b_{sg} \times t_s = 2 \times 95 \times 15 = 2850 \text{ mm}^2 \]

Derivation of \( p_c \)

\[ r_y = \sqrt{\frac{t_c}{(A_w + A_{sg})}} = \sqrt{\left( \frac{15 \times 1983}{12} \right) / (7.9 \times 316 + 15 \times 2 \times 95)} = 42.6 \text{ mm} \]

\[ L = D - 2T_c = 206.2 - 2 \times 12.5 = 181.2 \text{ mm} \]
\[ \lambda = \frac{0.7 \times L}{r_y} = \frac{0.7 \times 181.2}{42.6} = 2.98 \]

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

\[ p_c = 275 \text{ N/mm}^2 \]

\[ : P_c \text{ buckling} = (A_w + A_y) \times p_c = (2496 + 2850) \times 275 \times 10^3 \]

\[ = 1470 \text{ kN} \]

Crushing resistance of stiffener + column

\[ P_c \text{ crushing} = (A_{sn} \times p_y) + [(b_1 + n_2) \times t_c \times p_y] \]

where;

\[ A_{sn} = 2b_{sn} \times t_s = 2 \times (95 - 12) \times 15 = 2490 \text{ mm}^2 \]

\[ b_1 = \text{(calculated above)} = 54.9 \text{ mm} \]

\[ n_2 = \text{(calculated above)} = 113.5 \text{ mm} \]

\[ : P_c \text{ crushing} = [(2490 \times 275) + [(54.9 + 113.5) \times 7.9 \times 275]] \times 10^3 \]

\[ = 1051 \text{ kN} \]

Bearing resistance of stiffener alone

\[ P_c \text{ bearing} = \frac{A_{sn} \times p_{ys}}{0.8} = \frac{2490 \times 275 \times 10^3}{0.8} \]

\[ = 856 \text{ kN} \]

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

\[ M_1 = (F_{\text{min}} \times h_{\text{min}}) + (F_{r1} \times h_{r1}) \]

\[ = [530 \times (402.6 - 80 - 10.9/2) + 208 \times (402.6 - 60 - 10.9/2)] \times 10^3 \]

\[ = 252.9 + 70.1 = 323 \text{ kNm} \]
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**

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

where \( n_s = 2 \)

\( P_{ss} = 91.9 \text{kN from tables for an M20 bolt} \)

\( n_t = 2 \)

\( P_{ts} \) is the lesser of:

- 36.8 kN for bolt shear (from tables)
- 110 kN for end plate bearing
- 115 kN for column flange bearing

\( = 36.8 \text{kN} \)

\(:\text{Shear capacity } = 258 \text{kN} \) (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/mm² 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).

Worked example using the capacity tables

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

<table>
<thead>
<tr>
<th>Column (internal):</th>
<th>254x254x167 UC, grade S355</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams:</td>
<td>406x178x67 UB, grade S355</td>
</tr>
</tbody>
</table>

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).

<table>
<thead>
<tr>
<th>Beam Side</th>
<th>Column Side</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Moment capacity</strong></td>
<td>486kNm (page 86) &gt; 435kNm OK</td>
</tr>
<tr>
<td>High elongation reinforcement is not needed [there is no * in the table]</td>
<td></td>
</tr>
<tr>
<td>Beam is suitable in S355 only [capacity is given in bold type]</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical shear</strong></td>
<td>258kN (page 87) &lt; 400kN Unsatysfactory</td>
</tr>
<tr>
<td>without optional shear row</td>
<td></td>
</tr>
<tr>
<td>442kN (page 87) &gt; 400kN OK with optional shear row</td>
<td></td>
</tr>
<tr>
<td><strong>Column web panel shear</strong></td>
<td>Opposing beams give zero shear across the column web. OK</td>
</tr>
<tr>
<td>Tension zone: ✓ OK</td>
<td></td>
</tr>
<tr>
<td>Compression zone: ✓ OK</td>
<td></td>
</tr>
<tr>
<td>(no stiffening required - there is no S in the table)</td>
<td></td>
</tr>
</tbody>
</table>
### B.2 CAPACITY TABLES

**Contents**

**M20 bolted connections**

<table>
<thead>
<tr>
<th>End plate</th>
<th>Tension bolt rows</th>
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**M24 bolted connections**

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*Dimensions for detailing are shown on page 98.*
### Composite Connections

#### 1 ROW M20 8.8 BOLTS

<table>
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<tr>
<th>BEAM SERIAL SIZE</th>
<th>EFFECTIVE REINFORCEMENT (OPTION, NUMBER AND SIZE OF BARS, $A_{\text{reinf}}, F_{\text{reinf}}$)</th>
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<tr>
<td><strong>Effective Area</strong></td>
<td>$804 mm^2$</td>
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</table>

**Notes:**
- Beam may be either grade S275 or grade S355.
- Beam must be grade S355 to satisfy neutral axis position requirements.
- 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).

**Connection capacity:**
- Connection capacity exceeds 0.8 $M_p$ of composite beam in hogging (see Section 3.2.1 for significance of this).

**The value of $F_{\text{reinf}}$ 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.**

---

*Numbers in the table are illustrative and should be verified with the referenced tables and figures.*
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</tbody>
</table>

**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 $\Sigma F = F_{reinf} + F_{ Fr1 }$
S Provide compression stiffener.

---

Vertical Shear Capacity
258 kN without shear row
442 kN with shear row
**Composite Connections**

**2 ROWS M20 8.8 BOLTS**

**200 x 12 S275 END PLATE**

<table>
<thead>
<tr>
<th>BEAM Size</th>
<th>Effective reinforcement (option, number and size of bars, $A_{\text{reinf}}$, $F_{\text{reinf}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4 No. 16</td>
</tr>
<tr>
<td>B</td>
<td>6 No. 16</td>
</tr>
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<td>C</td>
<td>8 No. 16</td>
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<tr>
<td>D</td>
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</tr>
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<td>E</td>
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<tr>
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<td>6 No. 20</td>
</tr>
<tr>
<td>G</td>
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<td>H</td>
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<th>'A' mm</th>
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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 $F_{\text{fr}}$ 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.
### Appendix B

#### 2 ROWS M20 8.8 BOLTS

**200 x 12 S275 END PLATE**

<table>
<thead>
<tr>
<th>Panel Shear Cap. (kN)</th>
<th>Web Compr. Cap. (kN)</th>
<th>Tension Zone</th>
<th>Compression Zone</th>
<th>Column Serial Size</th>
<th>Re却ment option</th>
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**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 ΣF = F_reinf + F₁₁
- S: Provide compression stiffener.

---

**Diagram:**

- F_reinf.
- F₁₁ (208kN)
- F₂₂ (135kN)
- A
- ΣF
- B
- M
- V
- 515 kN
### Composite Connections

#### 2 ROWS M20 8.8 BOLTS
250 x 12 S275 END PLATE

<table>
<thead>
<tr>
<th>BEAM Serial Size</th>
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---

398 Beam may be either grade S275 or grade S355
369 Beam must be grade S355 to satisfy neutral axis position requirements
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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 $F_{\text{reinf}}$ 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.
## Appendix B

### 2 ROWS M20 8.8 BOLTS

250 x 12 S275 END PLATE

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<td>F₁₁</td>
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### 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 $\Sigma F = F_{\text{reinf}} + F_{11}$

S Provide compression stiffener.

![Diagram](image-url)
## Composite Connections

### 1 ROW M24 8.8 BOLTS

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<th>BEAM SERIAL SIZE</th>
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<th>BEAM SIDE</th>
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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 0.8Mpa of composite beam in hogging (see Section 3.2.1 for significance of this)

The value of \( F' \) 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

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#### 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 ℋF = F<sub>rein</sub> + F<sub>1</sub>
- S: Provide compression stiffener.

- Vertical Shear Capacity
  - 370 kN without shear row
  - 634 kN with shear row

---

F<sub>rein</sub>, F<sub>1</sub>, Vertical Shear Capacity
### Composite Connections

#### 2 ROWS M24 8.8 BOLTS

**200 x 15 5275 END PLATE**

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94
### 2 ROWS M24 8.8 BOLTS
200 x 15 S275 END PLATE

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**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 $\Sigma F = F_{\text{reinf}} + F_{T1}$
- S Provide compression stiffener.

**Notes:**
- Column capacity exceeds $\Sigma F = F_{\text{reinf}} + F_{T1}$
- Provide tension stiffener.
- Column satisfactory for bolt row tension values shown for the beam side.
- 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.

[Diagram of column and beam with shear capacity calculations]
### Composite Connections

**2 ROWS M24 8.8 BOLTS**

250 x 15 S275 END PLATE

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<tr>
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92

82

457x191x98

89

82

74

67

457x152x82

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67

60

52

398 Beam may be either grade S275 or grade S355
369 Beam must be grade S355 to satisfy neutral axis position requirements
364 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 \( F_{\text{reinf}} \) 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

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**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 $\Sigma F = F_{\text{refl}} + F_{r1}$
- Provide compression stiffener.
## Composite Connections

### B.3 DETAILING TABLES

**DIMENSIONS FOR DETAILING**

<table>
<thead>
<tr>
<th>Beam serial size</th>
<th>Dimension ( a_1 ) mm</th>
<th>Dimension ( a_2 ) mm</th>
<th>End plate overall depth ( D_F ) mm</th>
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<td>22 22</td>
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</table>

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

All plates to be S275.