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Design of Semi-Continuous Braced Frames

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FOREWORD

Semi-continuous construction may allow reduced beam depths or weights when compared with simple construction, whilst maintaining economy both in design effort and fabrication costs. This design guide, which was produced as part of the Eureka 130 CIMsteel project, is aimed at structural engineers and presents a method of analysis and design for steel frames which is suitable for hand (or computer) calculations. In developing the method, a conscious decision was made to keep the procedures as similar as possible to those associated with 'simple design'. The method is compatible with the requirements of BS 5950: Part 1, complying with part (a) of Clause 2.1.2.4.

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SUMMARY

In a semi-continuous frame the degree of continuity between the beams and columns is greater than that assumed in *simple design*, but less than that assumed in *continuous design*. The degree of continuity can be chosen to produce the most economic balance between the primary benefits associated with these two traditional alternatives.

This document presents a method of analysis and design which permits semi-continuous braced steel frames to be designed by hand. The method is only marginally more complex than that for *simple design*, and the connection details are straightforward (and therefore inexpensive). Connection forces and moments can be chosen so that column stiffening is not required. Despite this economy of both design effort and fabrication costs, when compared with *simple design*, it is possible to achieve:

- reduced beam depths
- reduced beam weights.

Procedures are given for checks at both the ultimate and serviceability limit states.

For normal design the practising engineer need only consult the main body of the document and the standard connection capacity tables given in Appendix C (yellow pages). A worked example of the approach is included in Appendix A. Appendix B gives a full procedure for estimating deflections more accurately, should this be required.

Dimensionnement de cadres contreventés à assemblages semi-continus

Résumé

Dans un cadre 'semi-continu', le degré de continuité entre les poutres et les poteaux est plus important que pour les cadres à assemblages 'simples', mais inférieur à celui rencontré dans les cadres dits 'continus'. Le degré de continuité peut être choisi pour obtenir la meilleure balance économique entre les avantages associés aux deux alternatives traditionnelles.

Cette brochure présente une méthode d'analyse et de dimensionnement qui permet un calcul manuel des cadres contreventés 'semi-continus'. Cette méthode n'est guère plus complexe que celle utilisée pour les cadres à assemblages simples. En plus, les détails d'assemblages ne sont guère compliqués et, dès lors, sont économiques. Par comparaison avec un dimensionnement basé sur des assemblages 'simples', on peut obtenir:

- *une réduction de la hauteur des poutres;*
- *une réduction du poids des poutres.*

Des procédures sont proposées pour la vérification à l'état ultime et en service.

Dans la plupart des cas habituels, le praticien doit seulement consulter la partie centrale du document et les tables donnant les capacités de résistance des assemblages standards, données à l'annexe C (pages jaunes). Un exemple complet

est donné à l'annexe A. L'annexe B donne une procédure permettant de calculer avec précision les flèches, si cela est nécessaire.

Berechnung von biegeweichen, unverschieblichen Tragwerken

Zusammenfassung

Bei einen biegeweichen Tragwerk ist die Steifigkeit des Anschlusses Träger-Stütze größer als bei einem Gelenk aber geringer als bei Annahme voller Tragfähigkeit. Der Grad der Biegeweichheit kann so gewählt werden, daß die wirtschaftlichste Lösung zwischen den beiden traditionellen Alternativen erreicht wird.

Dieses Dokument stelit eine Methode vor, die es erlaubt, biegeweiche, unverschiebliche Durchllauf- und Rahmentragwerke von Hand zu berechnen. Die Methode ist nur geringfügig aufwendiger als die 'einfache Berechnung' und die Anschlüsse sind einfach (und daher nicht teuer). Anschlußkräfte und Momente können so gewählt werden, daß eine Aussteifung der Stütze entfällt. Trotz der Einsparungen bei Berechnungsaufwand und Herstellungskosten kann gegenüber der 'einfachen Berechnung' folgendes erreicht werden:

- *geringere Trägerhöhen*
- *geringere Trägergewichte.*

Vorgehensweisen für Nachweise im Grenzzustand der Trag- und Gebrauchsfähigkeit werden aufgezeigt.

Für die Berechnung muß der Ingenieur nur den Hauptteil des Dokuments und die Tragfähigkeitstabellen für die Standard-Verbindungen im Anhang C (gelbe Seiten) zu Rate ziehen. Ein Berechnungsbeispiel befindet sich im Anhang A. Anhang B erlaubt eine genauere Abschätzung der Verformungen, falls dies nötig sein sollte.

Progettazione di Telai Controventati Semi-continui

Sommario

Nei telai semi-continui il grado di continuità tra le travi e le colonne risulta superiore a quello dei 'telai pendolari' e inferiore a quello dei 'telai a nodi rigidi'. Tale grado di continuità può essere selezionato in modo da raggiungere un conveniente equilibrio tra i benefici associati alle due tradizionali alternative progettuali ('telaio pendolare e telaio a nodi rigidi').

Questo documento propone un metodo di analisi e progetto per la progettazione manuale di telai semi-continui controventati. Il metodo è solo lievemente più complesso di quello normalment utilizzato per la progettazione di 'telai pendolari' e i dettagli del collegamento nei telai semi-continui si mantengono comunque semplici (e perciò poco costosi). Le forze e i momenti sul collegameno possono essere selezionate nella fase progettuale in modo che non siano richiesti irrigidimenti nella zone nodale della colonna.

In aggiunta alla convenienza economica, legata sia alla progettazione sia ai contenuti costi, i telai semi-continui controventati, se paragonati ai 'telai pendolari', consentono:

- *una riduzione dell'altezza delle travi*
- *una riduzione dei pesi delle travi.*

Nella pubblicazione vengono presentate le procedure di verifica agli stati limite sia ultimi sia di servizio.

Il corpo centrale del documento e le tabelle con le capacità portanti dei collegamenti riportate nell'Allegato C (pagine gialle) possono essere utilizzate per l'usuale progettazione. L'Allegato A propone un esempio applicativo mentre l'Allegato B presenta invece una procedura completa per la stima accurata della deformata del telaio, nel caso in cui questa sia richiesta.

Proyecto de estructuras aporticadas semi-continuas

Resumen

Como su nombre indica, en un pórtico semincontinuo el grado de continuidad entre vigas y columnas es mayor que el supuesto en los métodos simplificados ('simple design') pero inferior al supuesto en el proyecto continuo. El grado de continuidad puede escogerse para producir el mejor balance económico entre las ventajas asociadas con las dos alternativas tradicionales.

Este documento presenta un método de cálculo y proyecto que permite el diseño manual de estructuras aporticadas semicontinuas. El método tan solo es ligeramente más complicado que el simplificado y los detalles de uniones son inmediatos (y por tanto sin coste adicional). Las fuerzas y momentos en las uniones pueden escogerse de modo que no se precise la rigidización de las columnas.

Además de este ahorro en esfuerzo de proyecto y costes de fabricación, cuando se comparan las soluciones con las del método simplificado es posible conseguir:

- *cantos reducidos en las vigas*
- *peso reducido de las vigas.*

Se dan métodos de comprobación tanto para los estados límites últimos como de servicio.

Para casos normales el proyectista solo necesita consultar el núcleo del documento y las tablas de capacidad de uniones tipificadas incluidas en el Apéndice C (páginas amarillas). En el Apéndice A se incluye un ejemplo con detailles del método, mientras que en el Apéndice B se da un método para el cálculo más preciso de flechas cuando ello se estime necesario.

Dimensionering av stommar med elastiskt inspända anslutningar

Sammanfattning

I en elastiskt inspänd anslutning är graden av kontinuitet mellan balkar och pelare högre än vad som antas i en tedad anslutning, men lägre än vad som antas i en fast inspänd anslutning. Graden av kontinuitet kan varieras för att uppnå den mest ekonomiska balansen mellan de viktigaste fördelama med de båda traditionella metodema.

Denna publikation visar en dimensioneringsmetod och ett konstruktionsutförande som möjliggör en handberäkning av elastiskt inspända anslutningar. Metoden är endast något mer komplicerad än för ledade anslutningar, och

anslutningsdetaljema är enkla (och därigebom billiga). Tvärkrafter och moment kan väljas så att förstyvning av pelaren inte är nödvändig. Förutom ett ekonomiskt konstruktionsförfarande och utförande, jämfört med fritt upplagda anslutningar, är det möjligt att uppnå:

- *reducerade balkhöjder*
- *reducerad egentyngd hos konstruktionen.*

Anvisningama är redovisade för såväl brottgträns som brukdhränstillstånd.

Vid normal dimensionering behöver konstruktören endast utnyttja denna publikation inklusive Appendix C (gula sidor) som omfattar tabeller över bärförmåga för standarddetaljer. Ett beräkningsexempel finns i Appendix A. Appendix B ger anvisningar för att uppskatta nedböjningar mer noggrant, då detta krävs.

1 INTRODUCTION

Orthodox building frames generally comprise an assembly of beams and columns. The connections between the beams and columns are traditionally assumed to be either pinned, or able to achieve full moment continuity. Alternatively, it may be assumed that the connections transmit some moment, whilst permitting some relative rotation between the beam and column. The design and detailing of the connections, and the frame design method, must reflect the assumptions made about connection behaviour.

In BS 5950: Part 1[(1)], several methods of frame design are presented. Quoting from the code, these are:

Simple design

> *The connections between members are assumed not to develop moments adversely affecting either the members or the structure as a whole. The distribution of forces may be determined assuming that members intersecting at a joint are pin connected.....* (Clause 2.1.2.2)

Rigid design

> *The connections are assumed to be capable of developing the strength and/or stiffness required by an analysis assuming full continuity....* (Clause 2.1.2.3)

Semi-rigid design

> *Some degree of connection stiffness is assumed, but insufficient to develop full continuity as follows.*
>
> > *(a) The moment and rotation capacity of the joints should be based on experimental evidence....On this basis, the design should satisfy the strength, stability and stiffness requirements of all parts of the structure when partial continuity at the joints is to be taken into account in assessing moments and forces in the members.*
> >
> > *(b) As an alternative, in simple beam and column structures an allowance may be made for the inter-restraint of the connections between a beam and a column by an end restraint moment not exceeding 10 % of the free moment applied to the beam....* (Clause 2.1.2.4.)

Whilst *simple design* is used to design *simple structures*, *rigid design* is used for *continuous construction* (see BS 5950: Part 1, Section 5). No specific term is used in the code to describe frames for which *semi-rigid design* is appropriate. Clearly, this terminology is confusing, since the names given to the methods of design are not consistent with the types of construction to which they relate. Changes are likely in an amended version of BS 5950 (to appear in 1998).

For the purposes of this document, the following (rationalised) terminology has been adopted. The same names are given to both methods of design and types of construction:

- simple
- continuous
- semi-continuous.

Coupled with the lack of precision in terminology, BS 5950: Part 1 provides little guidance on the design of semi-continuous frames. The design procedures given in this document satisfy the requirements of BS 5950 Clause 2.1.2.4 (a).

The connections used in semi-continuous construction exhibit characteristics of partial strength, ductility, and either full or semi-rigidity. These terms are explained in Section 2.2.2.

1.1 Benefits of semi-continuous construction

Semi-continuous construction offers the following benefits for braced frames:

- beams may be shallower than in simple construction (this may be particularly advantageous, since it can ease service integration, allow a reduction in building height, and/or allow a reduction in cladding area)
- beams may be lighter than in simple construction
- connections are less complicated than in continuous construction
- frames are more robust than in simple construction.

Savings in beam weight and depth(2) are possible because of benefits at both the ultimate (ULS) and serviceability (SLS) limit states. The sagging moment which a beam must resist decreases as connection moment capacity increases. Connection stiffness means that the ends of a beam are restrained against rotation, so for a given deflection limit the bending stiffness of the beam can be reduced.

Disadvantages, compared with simple construction, are:

- an increase in connection cost (compared with the simplest of simple connections)
- a marginal increase in design complexity (although the procedures given in this guide remain essentially the same as for simple design).

Although outside the scope of this publication, unbraced frames may benefit even more than braced frames from semi-continuous construction. The connection characteristics enable wind loading to be resisted, without the extra fabrication costs incurred when full continuity is adopted. A design method for semi-continuous unbraced frames may be found in the SCI publication *Wind-moment design for unbraced frames*(3).

1.2 Scope of the publication

The design procedures given in this guide are applicable to frames with the following features:

- an orthogonal layout of beams
- bracing in both directions
- normal occupancy loading
- non-composite beams

- beams which are class 1 (plastic) or class 2 (compact)
- partial strength, semi-rigid (or rigid) connections to internal columns
- partial strength, semi-rigid (or rigid) connections to the major axis of perimeter columns
- simple connections to the minor axis of perimeter columns.

Each of these features is explained in more detail below.

Orthogonal layout of beams, braced in both directions

The method relies on the use of tried and tested connections. Details have not been developed for skew connections.

Bracing in both directions

Lateral loads must be resisted by bracing, not by frame action. Note that the standard partial strength, semi-rigid connections should not be subject to significant horizontal loads, and therefore may not be suitable for incorporation in the bracing system.

Normal occupancy loading

The method is not appropriate for buildings subject to storage loading, or dynamic loads.

Non-composite beams

The rules given are not appropriate for frames employing composite beams. However, the general philosophy of the method could be applied to such frames.

Class 1 or 2 beams

An ability to reach the plastic moment capacity is essential for the beams, which do not, however, need to be able to form plastic hinges.

Partial strength, semi-rigid connections

Partial strength connections enable hogging moments to be resisted at the beam ends. However, they can only be used when the support can resist the applied moment, namely for:

- connections to the flange of a column
- connections to the web of a column when there is an opposing beam with a connection of equal strength. This limitation is necessary unless the column is stiffened locally to prevent deformation of the web.

Partial strength connections must be ductile to ensure that they can behave as plastic hinges, as assumed in the design method. A lower limit on moment capacity (20% of that of the beam) is needed to avoid alternating plasticity in the connections. This limit also ensures that all perimeter columns are designed for some major axis moment (similar to the moments due to eccentric beam reactions in simple design). An upper limit is needed (50% of that of the beam) to ensure that the plastic hinges always occur in the connections rather than the beams. It also reduces the magnitude of moments applied to the columns (see Section 4.2.2).

Connections also need to possess a certain amount of stiffness, to reduce deflections at the SLS. They should be at least 'semi-rigid'.

Simple connections to the minor axis of perimeter columns

This limitation is needed unless local stiffening allows moment transfer into the column.

2 PRINCIPLES OF SEMI-CONTINUOUS DESIGN

2.1 Methods of analysis

The moments and forces in any (simple, semi-continuous or continuous) frame may be determined using an elastic analysis. Plastic analysis may alternatively be used[4], provided that the frame satisfies certain requirements, principally concerning ductility at potential plastic hinge locations.

Elastic analysis

In an elastic analysis, the stiffnesses of frame members are considered. Although widely used for simple and continuous frames, elastic analysis is not ideal for semi-continuous design because it requires quantification of connection stiffnesses, which may prove difficult in practice.

Plastic analysis

A plastic analysis considers the strengths of members and connections rather than their stiffnesses. Connection strength (moment capacity) can be predicted with sufficient accuracy using current methods. Plastic analysis is based on the assumption that plastic hinges form at critical points in the frame, and rotate to allow redistribution of moments. This rotation requires substantial ductility at these points[5].

Elastic-plastic analysis

In an elastic-plastic analysis, stiffness and strength considerations are both taken into account. Software may be used to perform this type of analysis for a semi-continuous frame, given knowledge of all the connection characteristics, i.e. stiffness (see comments above), strength and ductility. Elastic-plastic analysis is at present rarely used in design offices, although it is appropriate for certain types of structures such as portal frames.

2.2 Plastic frame analysis and design

2.2.1 Method

The plastic analysis and design procedures presented in this document for semi-continuous braced frames differ little from simple design according to BS 5950: Part 1. Figure 2.1 shows the internal moments in both a simple frame, and in a semi-continuous frame where plastic hinges have formed in the connections. The presence of these plastic hinges, which unlike simple connections have a significant moment capacity, means that the:

- beams are subject to smaller maximum (sagging) bending moments
- columns are subject to moments transferred by the connections, and therefore limited by the connection capacities, rather than nominal moments based on eccentric beam reactions.

Design of the beams is marginally more complex than in simple design, because the connection moment capacities must be included in a calculation of total moment capacity for comparison with the applied free bending moment. The presence of a hogging moment at each end of the beam means that the lower beam flange adjacent to each column is subject to compression over a short length. However, this length is generally sufficiently small to make a check of lateral torsional buckling in this region unnecessary. Critical lengths of beam for lateral torsional buckling can be calculated conservatively using BS 5950: Part 1 Clause 5.5.3.5.2. Reference may also be made to BS 5950: Part 1 Appendix G.

Column design considers moments based on the connection characteristics, rather than nominal moments calculated assuming eccentric beam reactions. Values of connection moment capacity for a standard range of connections are tabulated in Appendix C.

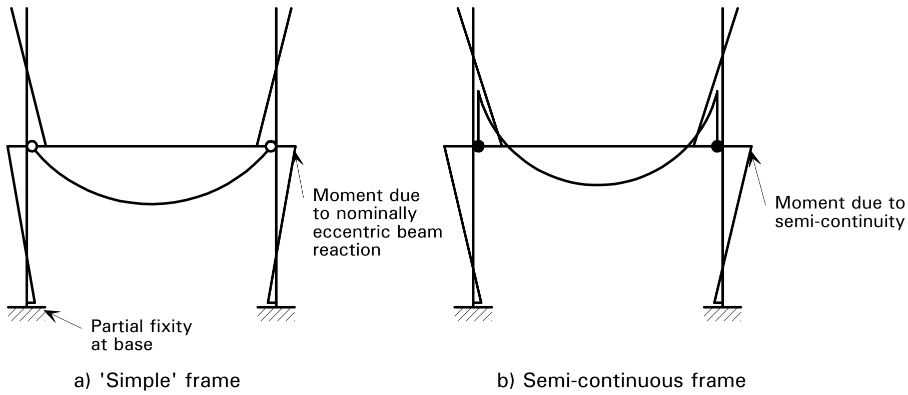

Figure 2.1 *Internal moments*

2.2.2 Connection characteristics

The behaviour of any type of connection may be fully described by a moment-rotation curve. The three most important characteristics which define such a curve are:

- stiffness, which is given by the slope of the curve
- strength (or moment capacity), which is given by the peak value of moment on the curve
- ductility, or rotation capacity, which is given by the maximum rotation which the connection can undergo before a significant loss in strength occurs. A connection which can undergo a rotation in excess of 0.03 radians is generally considered to be ductile[6].

These three characteristics are indicated in Figure 2.2, which shows the moment-rotation curve for a typical connection which might be used in semi-continuous construction.

The assumption made in plastic frame analysis and design, namely that plastic hinges form in the connections, requires the connections to be ductile enough to accommodate the necessary rotation without loss of strength. Connections suitable for use with the procedures given in this document must possess:

- strength (20% to 50% of the beam moment capacity)
- ductility (rotation at least 0.03 rad at failure)
- stiffness (enough to make them at least semi-rigid according to code definitions, for example Eurocode 3, Clause 6.4.2.3 [4]).

Although connection stiffness has no part to play in plastic analysis, it is worth noting that some stiffness is required to reduce deflections at the SLS.

Details of a standard range of connections possessing appropriate strength, ductility and stiffness, and therefore suitable for semi-continuous construction, are given in Section 3. Testing was used to demonstrate their ductility, and to quantify their strength and stiffness. The moment and shear capacity of these connections is tabulated in Appendix C.

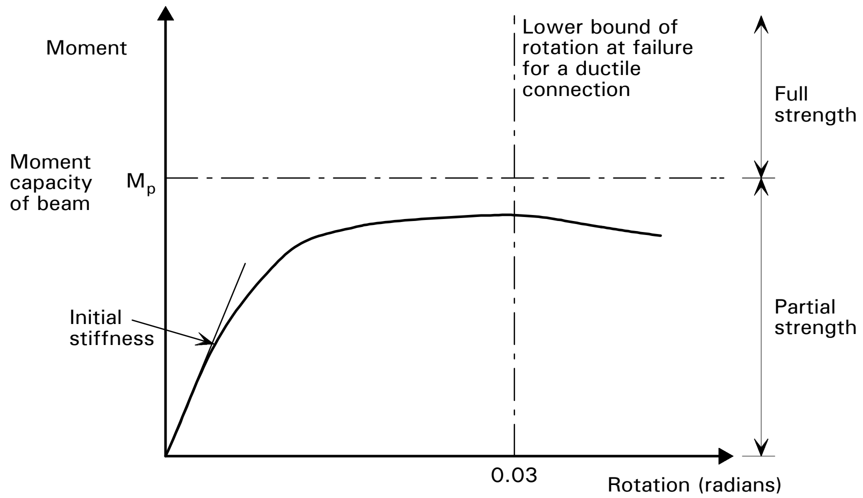

Figure 2.2 *Moment-rotation behaviour for a connection suitable for semi-continuous construction*

3 CONNECTIONS

3.1 End plate connections

The most practical type of connection that offers suitable characteristics for semi-continuous frames is the bolted end plate type. Although BS 5950[1] presents no rules for assessing connection ductility, Eurocode 3 (EC3)[4] states that end plate connections may be assumed to be ductile if the critical failure mechanism involves double curvature bending of the end plate or the column flange ('mode 1' failure). Most other failure mechanisms, such as those involving failure of the bolts or welds, or in the column compression zone, are non-ductile. Unfortunately, connection details which satisfy the EC3 requirement for mode 1 failure inevitably use thin end plates, and therefore possess limited strength and stiffness. An alternative approach for the designer, and one which may lead to more practical details, is to demonstrate connection ductility by testing.

3.2 Range of standard connections

For ordinary projects, it is usually neither practicable nor economic to test specific connection details. However, it is possible to use a range of standard details whose characteristics have been demonstrated by testing. A range of connections suitable for use in semi-continuous frames was developed at the SCI. A series of tests at the University of Abertay, Dundee[7], confirmed the characteristics of these connections. The standard connections have the following attributes:

- 12 mm thick (flush or extended) end plates when M20 bolts are used
- 15 mm thick (flush or extended) end plates when M24 bolts are used
- end plates fabricated from S275 steel
- full strength flange welds, with a minimum visible fillet of 10 mm
- continuous 8 mm fillet web welds.

These connections were originally developed for use in unbraced frames designed using the wind moment method. Because the wind loads on a frame may reverse, the connections in a wind moment frame need to be symmetrical so that they can resist both hogging and sagging moments[6]. Connections in a braced frame do not experience a reversal of moment, so the standard connections presented in Appendix C differ slightly from those given in Reference 6. Figure 3.1 illustrates a typical flush end plate connection for use in a braced frame.

The frame design procedures given in this document are based on the use of the standard range of connections presented in Appendix C. Other connection details providing similar strength, ductility and stiffness would be equally acceptable, however it should be noted that without testing it would be difficult to demonstrate the ductility that is essential for plastic analysis to be valid.

The weld sizes specified for the standard details are large relative to the end plate thickness, to ensure that failure of the welds does not occur. This restriction is necessary to avoid brittle failure of the connection. Modifying the weld sizes may have a significant influence both on the ductility and moment capacity of a connection.

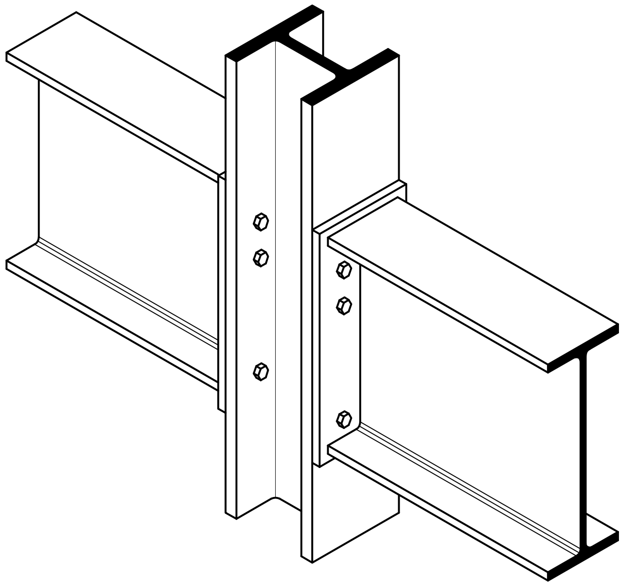

Figure 3.1 *Flush end plate connection*

Tables giving the moment and shear capacities of the standard connections are given in Appendix C. The values given in these tables are essentially the same as those found in Reference 6 for so-called ‘wind moment connections’. Reference 6 also contains the methodology used to derive the capacities. The tables in Appendix C also provide information concerning connection detailing. The designer should beware of varying the standard geometry, because dimensions, particularly those between bolt centrelines, are critical in many cases. Quite small changes could modify behaviour unacceptably.

The tables in Appendix C indicate whether a given column section size will require local stiffening when used with a given connection detail. Stiffening may be avoided by down-rating the connection strength, using information contained in the tables to calculate a revised capacity. Alternatively, stiffening may be avoided by choosing an alternative detail, or increasing the column size. The desire to avoid column stiffening arises because of the increased costs associated with the additional fabrication required.

4 DESIGN FOR THE ULTIMATE LIMIT STATE

4.1 Beams

In a semi-continuous braced frame, the required beam plastic section moduli are less than those required in an equivalent simple frame. This reduction is possible because of the partial strength nature of the connections. The weight, and/or depth of the beams can therefore be reduced. The reduction in required plastic section modulus is illustrated in Figure 4.1, which shows applied moments for a beam which is:

(a) simply supported at both ends

(b) simply supported at one end and semi-continuous at the other (with the important connection characteristic for the ULS being partial strength)

(c) semi-continuous at both ends.

The figure also shows schematically how the applied moments are related to the moment capacities of the beam and connections for design. The benefit of semi-continuous construction in reducing the sagging moment which the beam must resist is evident.

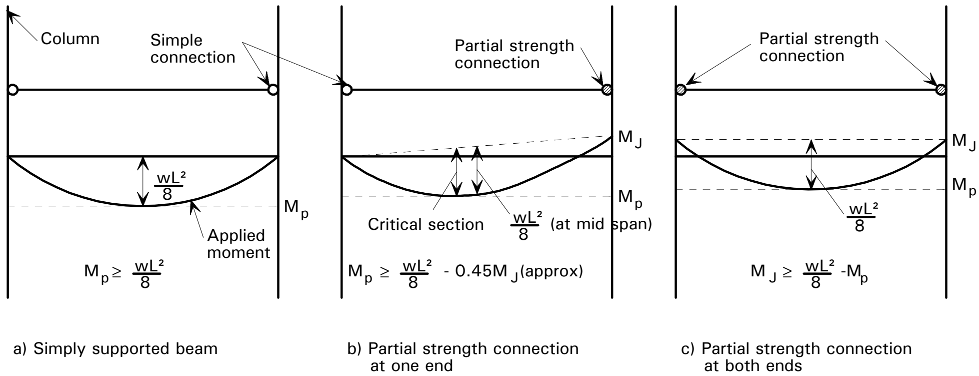

Figure 4.1 *Applied moments and moment capacities for beams with different support conditions*

4.2 Columns

4.2.1 Overall buckling check

Because moments are transferred from beams to columns in semi-continuous construction, it could be argued that to comply with BS 5950 Clause 5.1.2.1[1] there is a need to consider pattern loading. The code requires consideration of pattern loading for continuous construction to ensure that the loading arrangement which maximises the moments applied to a column is not more critical than the arrangement which maximises axial load (namely full loading on all beams). However, extensive testing and analyses[8,9] have demonstrated that for orthodox semi-continuous frames the most critical load pattern for overall column failure is always imposed load applied to all beams. The following two points in particular illustrate why this is so:

- The presence of a partial strength connection limits the moment that can be transferred from a beam when it is fully loaded. The connection effectively acts like a fuse to limit the moment which can pass through it.
- A column looses stiffness as it approaches collapse due to overall buckling. It therefore attracts less moment than predicted by a traditional elastic moment distribution.

The overall buckling check need only be performed, therefore, for dead plus imposed loading on all the beams. Internal columns need only be designed to resist unbalanced moment when it is due to differing connection strengths either side of a node. Unbalanced moment must also be considered in the overall buckling check of perimeter columns.

When choosing effective column lengths, the designer must make a reasonable assessment of the degree of restraint offered by the connection and base details he has chosen. Columns may be assumed to be effectively held in position and, because of the semi-rigid nature of the connections, partially restrained in direction (unless the column is particularly stiff relative to the beams and connections). The effective column length factor between 'semi-rigid restraints' may therefore normally be taken as 0.85, in accordance with BS 5950: Part 1 Table 24. For minor axis buckling, the designer must consider the restraint offered by the chosen connection and base details.

4.2.2 Local capacity check

A column approaching local failure is not subject to the same gradual loss of stiffness as one approaching overall buckling failure. It must therefore possess a local capacity which is sufficient to resist those moments predicted by an elastic distribution, including consideration of pattern loading.

However, columns can resist a greater axial load locally (resistance $A_g\, p_y$) than in overall buckling (resistance $A_g\, p_c$). A column which is fully utilised under axial load alone according to an overall buckling check therefore has some reserve to carry coincident moment locally. Additional reserve comes from the fact that there is less applied axial load under pattern loading than under full loading.

Numerical studies[9] have shown that for orthodox steel frames, local capacity is not likely to govern internal column sizes. For completeness, however, it is recommended that local capacity be checked under pattern loading at critical locations.

The maximum moment that can be applied to one side of the column is given by the connection moment capacity (M_j). The minimum opposing moment (M_d) can be taken as the built-in end moment due to factored dead load only, reduced by 35% to allow for connection and column flexibility[9].

The maximum unbalanced moment applied to the column is therefore M_j - M_d, to be distributed into the column lengths above and below the connections.

The local capacity of the column should therefore be checked in accordance with BS 5950: Part 1 Clause 4.8.3.2:

$$\frac{F}{A_g p_y} + \frac{M_x}{M_{cx}} \leq 1$$

where: F is the applied axial load (noting the reduction under pattern loading because imposed load is removed from one or more spans).

M_x is the proportion of unbalanced moment in the critical column length (normally equal to $(M_j - M_d)/2$, assuming identical column sizes and lengths above and below the connection).

The local capacity of perimeter columns should be checked in the normal way.

5 DESIGN FOR THE SERVICEABILITY LIMIT STATE

The following three responses of a braced frame at the Serviceability Limit State (SLS) may need to be checked:

- **deflections under imposed loads**, which may need to be limited to prevent damage to secondary elements (such as partitions, glazing or finishes) that are installed prior to the application of the imposed loads
- **deflections under total loads**, which may need to be checked to avoid impaired appearance of the building
- **vibrational response**, which may need to be checked to avoid unacceptable vibrations of the structure when it is subjected to dynamic loading from wind, or the movement of people etc.

The prediction of how a frame will deflect under loading is not an exact science. Also, whether or not a given deflection is acceptable is a subjective matter. Taking both these points into consideration, although recommended limits for deflections under certain loading conditions are given in codes[1,4], different values may be chosen and agreed if appropriate. A degree of approximation in calculating values can certainly be justified.

The designer should consider why deflection limits are specified, and what loading conditions are critical. For example, glazing panels will probably be fitted after application of the majority of the dead load (i.e. after casting of the concrete floor slabs), but before application of any imposed loads. Subsequent deflections of the structure as imposed loads are applied, including any deflections due to inelastic deformation of the connections, will therefore need to be accommodated by the glazing. As far as the glazing is concerned, deflection of the structure under imposed loads is therefore critical and should be controlled, whereas total deflection is unimportant.

5.1 Deflection under imposed load

For calculating deflections, beams should be thought of as being rotationally restrained at the supports by springs (see Figure 5.1). The spring stiffness represents the stiffness of the connection itself, plus that of the adjoining structure. Because of this stiffness, beam behaviour lies between 'built-in' and 'simply supported'.

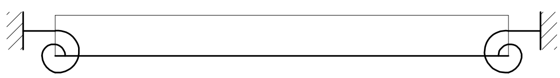

Figure 5.1 *Beam model for deflection*

For the analysis of orthodox frames (see scope in Section 1.2), there is no need for the designer to determine an effective spring stiffness provided the standard connections listed in Appendix C are used. The following formulae were derived using the procedures given in Appendix B, considering appropriate values of connection and member stiffnesses:

Uniformly distributed loading

$$d_{\text{imposed}} = \frac{b}{384}\frac{wL^4}{EI}$$

For an internal span:

- with connections having a partial strength in excess of 45%, $\beta = 3.0$
- with connections having a partial strength less than 45%, $\beta = 3.5$

For an external span:

- with connections having a partial strength in excess of 45%, $\beta = 3.5$
- with connections having a partial strength less than 45%, $\beta = 4.0$

Point loads at third points

The following deflection coefficients should be used for a beam with a point load of magnitude P at each of its third points (total beam load $3P$):

$$d_{\text{imposed}} = \frac{b}{648}\frac{PL^3}{EI}$$

For an internal span:

- with connections having a partial strength in excess of 45%, $\beta = 14$
- with connections having a partial strength less than 45%, $\beta = 17$

For an external span:

- with connections having a partial strength in excess of 45%, $\beta = 16$
- with connections having a partial strength less than 45%, $\beta = 19$

Deflection coefficients are expressed as multiples of 1/384 or 1/648 so that reductions from simply supported values are evident. Reference should be made to the full procedure given in Appendix B for other load configurations.

External span values assume a pinned connection at one end of the beam (i.e. they represent an extreme case). Internal span values are for beams with equal end connections. For situations between these two extremes, as will often occur in practice, linear interpolation may be used to determine the appropriate deflection coefficient.

5.2 Deflection under total load

If total load deflection needs to be checked, imposed load deflections determined using the procedure given in Section 5.1 must be added to dead load deflections. Dead load deflections may be calculated using the same procedure.

5.3 Vibrational response

Beams and slabs subjected to rhythmic loading may vibrate and thereby affect either occupant comfort or the performance of secondary building elements such as partitions. Resonance may occur if the natural frequency of the members corresponds to the excitation frequency.

To assess whether the response of a beam to dynamic loads needs investigating, the designer should check the natural frequency of the beam. Natural frequency can be calculated as a function of deflection under dead load[10]. All beams should be taken as having simple supports for the purposes of this check, which is simply a means of assessing stiffness:

$$f_{\mathrm{i}} = \frac{18}{\sqrt{d_{\mathrm{dead}}}}$$

where δ_{dead} is in mm, and calculated assuming simple supports (see note below).

For normal buildings, this frequency should exceed 3 Hz according to Reference 10. For buildings to be used for rhythmic group activities such as dancing, the limit is 5 Hz.

If the beam's natural frequency does not exceed the chosen limit, floor response must be predicted. Full procedures for doing so are given in Reference 10. Although connection stiffness is not allowed for in the calculation of f_{i}, which is based on the deflection of a simply supported beam, the semi-continuity between members improves the response of the structure.

6 DESIGN PROCEDURES

The following principal steps define the design procedure at the ULS and the SLS for a semi-continuous braced frame falling within the scope of this document.

6.1 Scheme design

Columns

Select column sizes to resist axial load alone in an overall buckling check. The utilisation of perimeter columns should be limited to 0.8, to allow some reserve for applied moment. The utilisation of internal columns may approach 1.0. These utilisation limits should be modified if:

- internal columns will be subject to unbalanced moment as a result of unequal connection strengths
- differing spans will lead to significant minor axis moments (which are calculated as in simple design, assuming eccentric beam reactions).

Beams

Select class 1 or 2 beam sizes, based on the following criteria:

- Internal span $M_p \approx 0.70\ M_o$
- External span $M_p \approx 0.80\ M_o$

M_p = moment capacity of the beam
M_o = free bending moment at the ULS

6.2 Final design

Connections

Select standard connections from the design tables in Appendix C. The minimum connection moment capacity must satisfy the shortfall between the maximum applied moment and the moment capacity of the beam. Doing so means that no further check of the beams is required for the ULS.

The connection moment capacity should not exceed 50% of the beam capacity for a connection to an internal column.

The moment capacity of a connection to an external column should be approximately 20% of the beam capacity.

Connections should ideally be chosen to avoid the need for column stiffeners (see Appendix C). To achieve this it may be more economic to choose a connection with a smaller moment capacity, possibly necessitating a heavier beam, or to increase the column size.

Check the connection shear capacity using the tables in Appendix C, and add 'shear bolts' if necessary.

Beams

Calculate beam deflections under imposed (unfactored) loading, using appropriate formulae and deflection coefficients from Section 5.1. Check the calculated values against appropriate limits (span/200 generally, or span/360 if deflections will damage brittle components, according to BS 5950: Part 1).

If excessive total deflections would impair appearance, calculate dead load deflections using the procedures given in Section 5.1. Add these deflections to those under imposed load. Total deflections should be compared with an appropriate limit. Although EC3 suggests span/250, a less onerous requirement may be appropriate in many cases.

Compare natural and excitation frequencies to determine whether floor response to dynamic loading needs to be assessed (see Section 5.3). If necessary, check the floor response using the procedures given in Reference 10.

Columns

Check internal columns for overall buckling under the applied axial load in combination with any moment about the major axis resulting from unequal connection strengths, and any unbalanced minor axis moments. Minor axis moments should be calculated and distributed as in simple design, assuming eccentric beam reactions. The internal columns should also be checked for local capacity, considering axial loads and moments under pattern loading. Use the simplified procedure given in Section 4.2.2.

Check perimeter columns for the applied axial load in combination with any major or minor axis moments. Both overall buckling and local capacity checks are required.

Check that the column sizes identified in the final design are compatible with the connection details, preferably without the need for column stiffening.

Details

Design the column bases, column splices, and the frame bracing systems as in ‘simple construction’. The detailing of bases and splices, which may be pinned or chosen to provide moment continuity, must be properly reflected in the frame analysis and design assumptions.

Care should be taken if standard connections are used as part of the bracing system, because the behaviour of the connections may be adversely affected by the presence of additional axial or shear loads in the beams, or detailing to accommodate the bracing members.

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APPENDIX A Worked example

The following worked example considers a two storey frame. It is assumed that the primary reason for using semi-continuous construction in this instance is to reduce the depth of the beams. Connections providing a high degree of partial strength are therefore chosen, which result in a requirement for column stiffeners. The increase in fabrication cost (or material cost if heavier columns are chosen to obviate the need for stiffeners) is assumed to be acceptable in this context.

The effectiveness of semi-continuous construction in permitting significant savings in beam depth and/or weight can be appreciated by comparing beam sizes in the table below. The results of this worked example are given alongside the beam sizes which would be required for simple construction.

	Semi-continuous design	**Simple design**	**Saving in beam depth**	**Saving in beam weight**
Floor beam Internal Span	457 × 191 × 89	533 × 210 × 109	76 mm	20 kg/m
Floor Beam End Span	457 × 191 × 98	533 × 210 × 109	72 mm	11 kg/m
Roof Beam Internal Span	356 × 171 × 45	406 × 178 × 54 or 356 × 171 × 67	51 mm	22 kg/m
Roof Beam End Span	356 × 171 × 45	406 × 178 × 54 or 356 × 171 × 67	51 mm	22 kg/m

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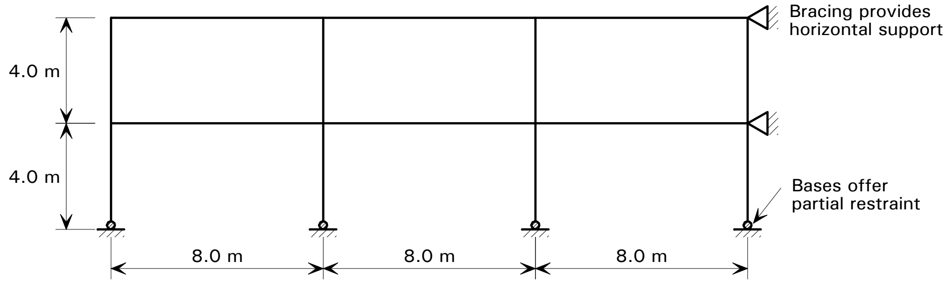
CALCULATION SHEET

Job No: *BCC4922*		Page *1* of *11*	Rev
Job Title *Semi-continuous braced frames*			
Subject *Worked example*			
Client *CIMsteel*	Made by *GHC*	Date *Mar 1997*	
	Checked by *DGB*	Date *Mar 1997*	

1. INPUT

1.1 Geometry



Building assumed to have 2 storeys with an inter storey height = 4.0 m. Column size does not vary between storeys

Beam span = 8.0 m

Frame centres = 6.5 m

Loaded area per beam = 8.0 × 6.5 = 52.0 m²

1.2 Loading

Floor dead load
$= 4.5\ kN/m^2$

Floor live load
$= 5.0\ kN/m^2$

Ultimate load at floor level
$= 1.4 \times 4.5 + 1.6 \times 5.0 = 14.3\ kN/m^2$

Roof dead load
$= 3.0\ kN/m^2$

Roof live load
$= 0.75\ kN/m^2$

Ultimate load at roof level
$= 1.4 \times 3.0 + 1.6 \times 0.75 = 5.4\ kN/m^2$

2. SCHEME DESIGN

2.1 Internal Column

Axial load on column = 52.0 × (14.3 + 5.4) = 1024 kN

Assume that the base detail and the major and minor axis beam connections provide partial directional restraint to the column, so that the effective length factors are 0.85 for buckling about both axes (BS 5950: Pt 1, Table 24).

Effective length = 0.85 × 4.0 = 3.4 m

From member capacity tables (e.g. SCI publication P202 4th edition) for Universal Columns subject to axial load (BS 5950 Clause 4.7.4), for S275 steel and an effective length of 3.4 m (requiring interpolation) try 203 × 203 × 46 UC,

P_{cx} = *1478 kN*

P_{cy} = *1112 kN*

Utilisation=1024/1112=92% (utilisation should not exceed 100% for a regular frame)

Pass

2.2 External Column

Axial load on column = 0.5 × 52.0 × (14.3 + 5.4) = 512 kN

Assume an effective length factor of 0.85 (see note above)

Effective length = 3.4 m

Universal columns smaller than 203 × 203 will not be used, due to potential difficulties forming beam connections with smaller columns.

From tables (SCI publication P202 4th edition), for S275 steel and an effective length of 3.4 m, try a 203 × 203 × 46 UC,

P_{cx} = *1478 kN*

P_{cy} = *1112 kN*

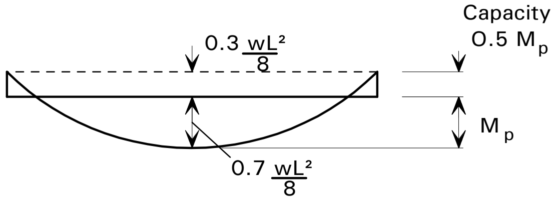
Note that the utilisation = 512/1112 = 46% (utilisation at scheme design stage should not exceed 80% for an external column in a regular frame)

Pass

2.3 Floor Beams - Internal Span

A connection having a strength equal to approximately 50% of the beam moment capacity will be used at each end of the span, therefore the maximum applied sagging moment is taken as 70% of the free moment (roughly twice the support moment):

Free Moment = $6.5 \times 14.3 \times 8.0^2/8 = 744$ kNm



Applied sagging moment = $0.7 \times 744 = 521$ kNm

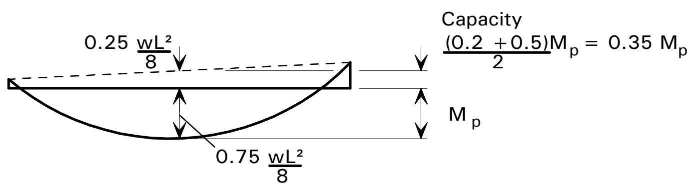
From tables (SCI publication P202 4th edition) try 457 × 191 × 89 UB in S275 steel

Moment capacity (assuming full lateral restraint to top flange) $M_{cx} = 534 > 521$ kNm

Pass

2.4 Floor Beams - End Span

The connection to the external column will have a strength equal to approximately 20% of the beam moment capacity. The connection to the internal column will have a strength equal to approximately 50% of the beam capacity. The applied sagging moment is therefore taken as 75% of free moment:



Free Moment = $6.5 \times 14.3 \times 8.0^2/8 = 744$ kNm

Applied sagging moment = 0.75 × 744 = 558 kNm

From tables (SCI publication P202 4th edition) try 457 × 191 × 98 UB in S275 steel

Moment capacity = M_{cx} = 592 > 558 kNm

Pass

2.5 Roof Beams - Internal Span

Calculation as 2.3, chosen section is 356 × 171 × 45 UB, S275 (213 > 197 kNm)

2.6 Roof Beams - End Span

Calculation as 2.4, chosen section is 356 × 171 × 45 UB, S275 (213 > 211 kNm)

3. FINAL DESIGN

3.1 Connections

3.1.1 Floor Beams - Internal Span Connections

The connection moment capacity must be compatible with the beam design assumptions, namely the connection capacity must satisfy the difference between the free bending moment and the beam capacity. An upper limit of 50% of the beam moment capacity should also be respected (see Section 1.2 of this design guide).

Chosen beam is a 457 × 191 × 89 UB

Minimum required connection capacity
= free moment - beam capacity
= 744 − 534 = 210 kNm

Maximum allowable connection capacity
= 50% of beam moment capacity
= 0.5 × 534 = 267 kNm

From the tables of standard moment connections (Appendix C page 56) for a 457 × 191 beam, connection moment capacity for 2 rows of M24 tension bolts, with an extended 200 × 15 mm end plate, is 213 kNm.

Check against minimum connection requirement: 213 > 210 kNm

Check against maximum connection allowable: 213 < 267 kNm

<u>***Beam and Connection OK***</u>

3.1.2 Floor Beams - End Span Connections

For the connection to the internal column

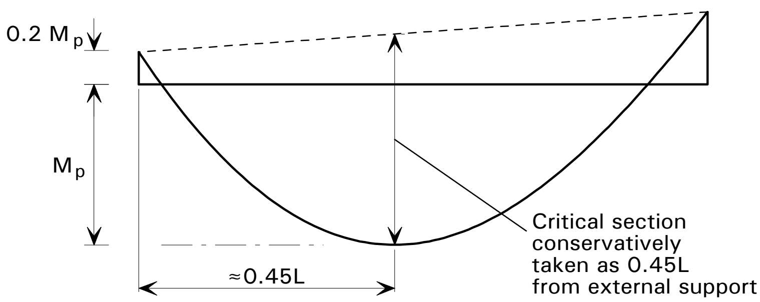
Chosen beam is a 457 × 191 × 98 UB

Assuming an external connection with a capacity (M_{j1}) equal to 20% of the beam moment capacity, the minimum internal connection capacity (M_{j2}) can be determined.

At the critical section (conservatively assumed to be at 0.45L from the weaker (external) connection, since the exact position will be between 0.45L and 0.5L):

total capacity $\approx M_p + M_{j1} + 0.45(M_{j2} - M_{j1})$

where M_p *is the moment capacity of the beam,*
M_{j1} *is the capacity of the weaker connection,* $= 0.2\ M_p$,
M_{j2} *is the capacity of the stronger connection.*



total capacity $= 592 + 0.2 \times 592 + 0.45(M_{j2} - 0.2 \times 592)$

free moment = 744 kNm

therefore $744 \leq 1.2 \times 592 + 0.45\ (M_{j2} - 0.2 \times 592)$

$M_{j2} \geq (744 - 1.11 \times 592)/0.45 = 193$ *kNm*

Maximum connection capacity $= 50\%$ *of beam moment capacity*
$= 0.5 \times 592 = 296$ *kNm*

From the tables of standard moment connections (Appendix C page 56) for a 457 × 191 beam, connection moment capacity for 2 rows of M24 tension bolts, with an extended 200 × 15 mm end plate, is 213 kNm.

Check against minimum connection requirement: 213 > 193 kNm

Check against maximum connection allowable: 213 < 296 kNm

For the connection to the external column, a moment capacity (M_{j1}) of approximately 20% of the beam capacity has been assumed. From tables (Appendix C page 47), connection moment capacity for 2 rows of M20 tension bolts, with a flush 200 × 12 mm end plate, is 123 kNm

Minimum required connection capacity $= 0.2 \times 592 = 118$ *kNm*

Capacity = 123 kNm > 118 kNm

Beam and Connections OK

3.1.3 Roof Beams - Internal Span Connections

Calculation as 3.1.1, chosen connection uses 2 rows of M20 tension bolts, with an extended 200 × 12 mm end plate. Connection moment capacity = 107 kNm (page 49)

3.1.4 Roof Beam - End Span Connections

Calculation as 3.1.2, chosen internal connection uses 2 rows of M20 tension bolts, with an extended 200 × 12 mm end plate. Connection moment capacity = 107 kNm (page 49)

External connection adopts 1 row of M20 tension bolts, with a flush 200 × 12 mm end plate. Connection moment capacity = 60 kNm (page 46).

3.2 Beams (check serviceability)

3.2.1 Floor Beams - Internal Span

Beam chosen from scheme design is 457 × 191 × 89 UB

Deflection under imposed load

Serviceability imposed load at floor level = 1.0 × 5.0 = 5.0 kN/m²

Imposed load deflection (conservatively taking the connection strength as less than 45%, which corresponds to a relatively flexible connection - see Section 5.1)

$= \quad 3.5 \times (5.0 \times 6.5) \times 8.0^4 / (384 \times E \times 41020) = 14.1 \text{ mm}$

Allowable deflection under imposed load (assuming brittle finishes would be damaged by excess deflection)

$= \quad 8000/360 = 22.2 \text{ mm}$

Assume that total load deflections do not need checking, because they will not impair appearance.

Check: 14.1 < 22.2 mm

<u>***Deflections OK***</u>

Vibration response

Serviceability dead load at floor level = 1.0 × 4.5 = 4.5 kN/m²

Dead load deflection of the beam assuming simple supports

$= \quad 5 \times (4.5 \times 6.5) \times 8.0^4 / (384 \times E \times 41020) = 18.0 \text{ mm}$

Natural frequency of the beam

$= \quad 18 / \sqrt{18.0} = 4.2 \text{ Hz}$

This exceeds the lower limit of 3 Hz, so a check of beam response to dynamic loading is not required.

3.2.2 Floor Beams - End Span

Calculation as 3.2.1.

Imposed load deflection

$$= \quad 4 \times (5.0 \times 6.5) \times 8.0^4 / (384 \times E \times 45730) = 14.5 \text{ mm}$$

Check: 14.5 < 22.2 mm

Deflections OK

3.2.3 Roof Beams

Calculation as 3.2.1

Deflections OK

Vibration check is not needed at roof level.

3.2.4 Roof Beams - End Span

Calculation as 3.2.1

Deflections OK

Vibration check not needed.

3.3 Columns

3.3.1 Internal Columns

Because the frame is regular, internal columns are not subject to unbalanced moment from unequal strength connections.

Note: ***The scheme design may need to be refined (depending on the utilisation calculated at the scheme design stage) to allow for a slightly increased axial load in the internal column adjacent to the end span. This increase is due to moment gradient in the end span, but has not been included for the overall buckling check in this example for the sake of brevity. For this particular case, the calculated axial load would increase from 1024 kN to 1041 kN (an increase of 1.7%).***

Local capacity check (BS 5950, Clause 4.8.3.2 (a)) for column length between the base and the first storey (using the procedure given in Section 4.2.2)

$M_j = 213$ kNm

$M_d = (1 - 0.35) \times 6.5 \times 1.4 \times 4.5 \times 8.0^2 / 12 = 142$ kNm

Applied moment (assuming connection moment distributes evenly above and below the beam) = (213 – 142) / 2 = 35.5 kNm

Reduced axial load (no imposed load on one span)
= 1024 – 1.6 × 5 × 6.5 × 8 = 608 kN

From tables (SCI publication P202 4th edition), for a 203 × 203 × 46 UC

$A_g\, p_y \quad = \quad 1620$ kN

$M_{cx} \quad = \quad 137$ kNm

$$\text{Interaction} \quad = \quad \frac{608}{1620} + \frac{35.5}{137} = 0.38 + 0.26 = 0.64$$

By inspection, the column length between the first and second storeys is not critical

<u>*Local capacity check is OK*</u>

The connection design tables given in Appendix C should be used to determine if the chosen column requires local stiffening.

Column is 203 × 203 × 46 UC

From 3.1.1: chosen connection is from the table on page 56, for a 457 × 191 × 89 beam. The 'column side' information indicates that the chosen column section has insufficient capacity in the tension zone. This can be resolved by increasing the column size, or local stiffening (strengthening) of the column.

From 3.1.3: chosen connection is from the table on page 49, for a 356 × 171 × 45 beam. The 'column side' information indicates that the chosen column section has insufficient capacity in the tension zone. This can be resolved by increasing the column size, or local stiffening (strengthening) of the column.

3.3.2 External Columns

External columns are subject to unbalanced loading, therefore major axis bending must be considered in combination with axial load.

Beam reactions (allowing for moment gradient) are:

At floor beam $= (14.3 \times 6.5 \times 0.5 \times 8) - 213/8 + 123/8$
$= 361$ *kN*

At roof level $= (5.4 \times 6.5 \times 0.5 \times 8) - 107/8 + 60/8$
$= 135$ *kN*

Axial load in column $= 361 + 135 = 496$ *kN*

Column chosen from scheme design is 203 × 203 × 46 UC, S275

Overall buckling check (BS 5950, Clause 4.8.3.3.1) for column length between the base and the first storey

Applied moment (assuming connection moment distributes evenly above and below the beam) = 123/2 = 61.5 kNm

Conservatively assuming a pinned base, m = 0.57

From tables (SCI publication P202 4th edition), for a 203 × 203 × 46 UC with an effective length of 3.4 m (requiring interpolation)

$A_g \, p_{cy} = 1112$ *kN*

$M_b = 119$ *kNm*

Interaction $= \dfrac{496}{1112} + \dfrac{0.57 \times 61.5}{119} = 0.45 + 0.29 = 0.74$

By inspection, the column length between the first and second storeys is not critical

Combined buckling check is OK

Local capacity check (BS 5950, Clause 4.8.3.2 (a)) for column length between the base and the first storey

Applied moment = 61.5 kNm

Axial load = 496 kN

From tables (SCI publication P202 4th edition), for a 203 × 203 × 46 UC

$A_g \, p_y = 1620$ *kN*

M_{cx} = *137 kNm*

Interaction = $\frac{496}{1620} + \frac{61.5}{137} = 0.31 + 0.45 = 0.76$

By inspection, the column length between the first and second storeys is not critical

<u>Local capacity check is OK</u>

Use Appendix C to check whether column requires local stiffening - procedure as outlined in 3.3.1.

APPENDIX B Deflection calculations

Coefficients for calculating deflections under imposed and dead load are given in Section 5 of this guide. In this appendix, a full procedure for calculating deflections is given, to be used when a more accurate calculation of deflections is needed (the coefficients in Section 5 are conservative, based on assumed 'support' stiffness).

B.1 General principles

Beams are assumed to be restrained by springs which model the presence of the connections and attached columns and adjacent beams (see Figure 5.1). These springs provide a support which lies between 'built-in' and 'simply supported'.

The consequence for design of some support flexibility is that for a given load the hogging moments at the beam ends are lower than they would be for a built-in beam. Sagging moments are consequently higher. The redistribution of hogging moment into the span increases as connection stiffness decreases, and is 100% (zero support moment) for the zero support stiffness associated with a simply supported beam. This can be thought of in a different way. A certain percentage of the imposed load can be considered as being applied to a built-in beam, and the remainder applied to a simply supported beam. This model is not only applicable to moments; deflections can also be calculated for each of the two cases (simply supported and built-in), and summed.

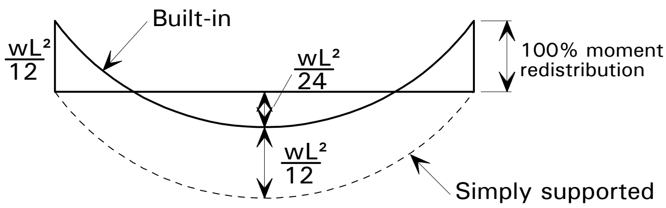


Figure B.1 *Redistribution of bending moments*

Figure B.2 shows the deflection coefficient β as a function of 'support'/beam stiffness, for an internal span subject to Uniformly Distributed Loading (UDL). The formula for deriving deflection from β is shown in the figure. The curve shown can be used to calculate deflections once the designer has determined the relative 'support'/beam stiffness for a particular case (the exact definition of 'support', as well as procedures for calculating relative stiffness, are given in Section B.2).

Zero 'support' stiffness represents the case of a simply supported beam. This corresponds to a value of β equal to five for UDL ($\delta = 5wL^4/384EI$). As the relative support stiffness increases, and the beam tends towards being built-in, the curve approaches a horizontal asymptote at β equal to one ($\delta = 1wL^4/384EI$).

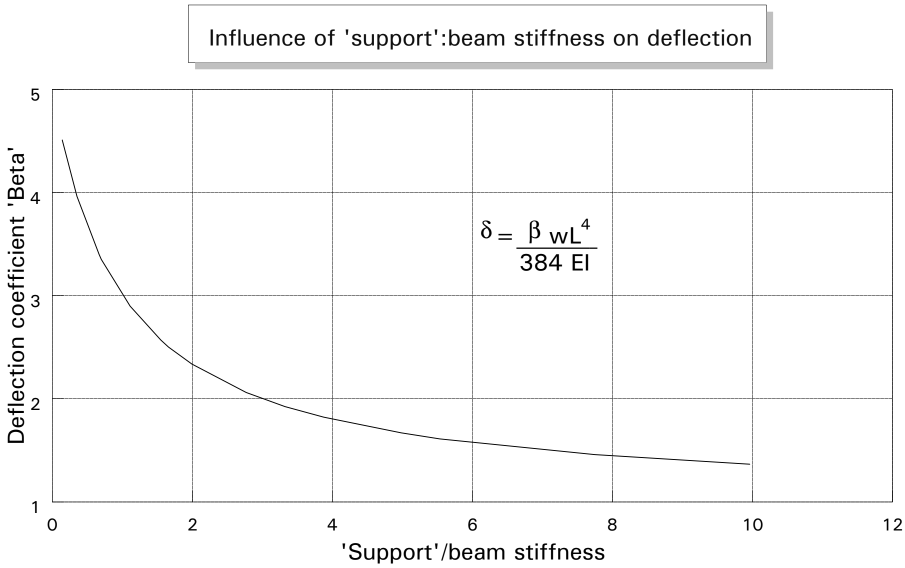


Figure B.2 *Deflection as a function of relative stiffness - internal span with UDL*

Figure B.3 is essentially the same as Figure B.2, except that the ordinate shows moment redistribution rather than β. It can be seen that a relative stiffness of zero corresponds to a redistribution value of one (or 100%). The theoretical end moments determined assuming built-in supports would therefore be completely redistributed for this case, giving zero end moments (which is correct for a simply supported beam). For a stiff support, redistribution approaches zero, which is correct for a built-in beam.

The following relationship exists between redistribution and β for an internal span with UDL:

$$\beta = 1 + 4(\text{redistribution})$$

so that when the redistribution is one (simply supported), β equals 5.0. When the redistribution is zero (built-in), β equals 1.0. The moment redistribution scale on Figure B.3 can be used to derive deflection coefficients for different beam and load arrangements, using the following relationships:

- internal span with point load at mid span $\quad \beta = 2 + 6$ (redistribution)
- external span with UDL $\quad \beta \approx 2 + 3$ (redistribution)
- external span with point load at mid span $\quad \beta \approx 3.5 + 4.5$ (redistribution)

'External span' here describes the extreme case of a beam which is simply supported at one end. Similar relationships for other situations can be derived knowing that when the redistribution equals one, the deflection must be that of a simply supported beam, and when the redistribution equals zero, the deflection must be that of a built-in beam.

The derivation of the design curves given in Figures B.2 and B.3 is described, for information, in Section B.3 of this Appendix.

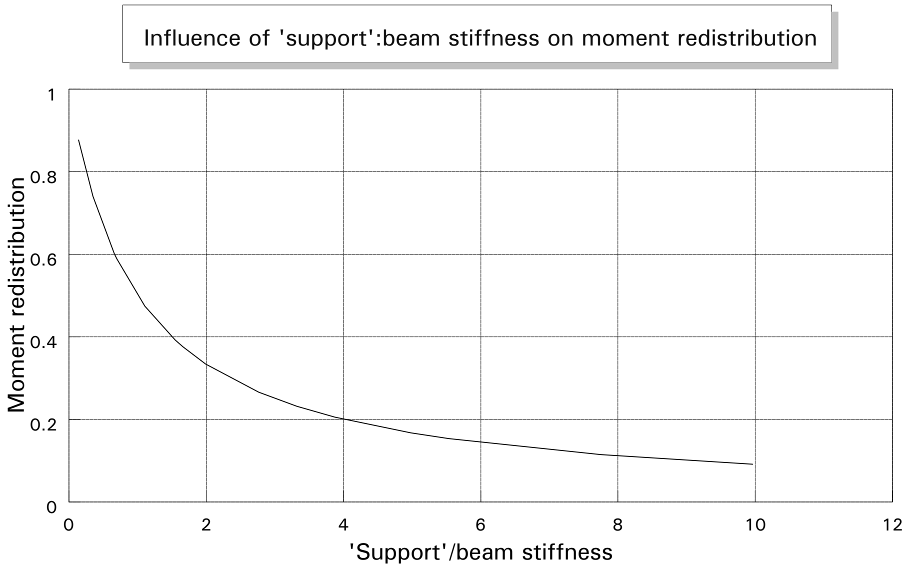


Figure B.3 *Moment redistribution as a function of relative stiffness*

B.2 Relative stiffness

In order to use Figure B.2 (or Figure B.3 where loading or framing arrangement dictate), the designer must calculate the stiffness of the 'support' relative to the stiffness of the beam being checked.

'Support' stiffness

The 'support' stiffness may be influenced by:

- the stiffness of the connection itself
- the apparent stiffness of the column to which the connection is attached.

To illustrate these two points, consider the subframe shown in Figure B.4. In this figure, the 'support' to the left hand end of Beam 2 comprises Connection 21, plus Column 1 and Column 2, and Beam 1. The deflection of Beam 2 is affected by the stiffnesses of all these components. The apparent stiffness of the column is a function of Column 1, Column 2, and Beam 1.

Suitable values of **connection stiffness** (k_j) can be derived from test results, or tabulated values. Initial stiffness is generally appropriate for the calculation of dead load deflections, but the connections may enter into the elasto-plastic or even plastic regions of response as imposed loads are applied (see schematic connection behaviour shown in Figure B.5). The designer must consider levels of applied moment in order to determine an appropriate stiffness for each stage of load application. When neither test results nor tabulated information are available, the procedures given in EC3 Annex J may be used to calculate connection stiffness[4].

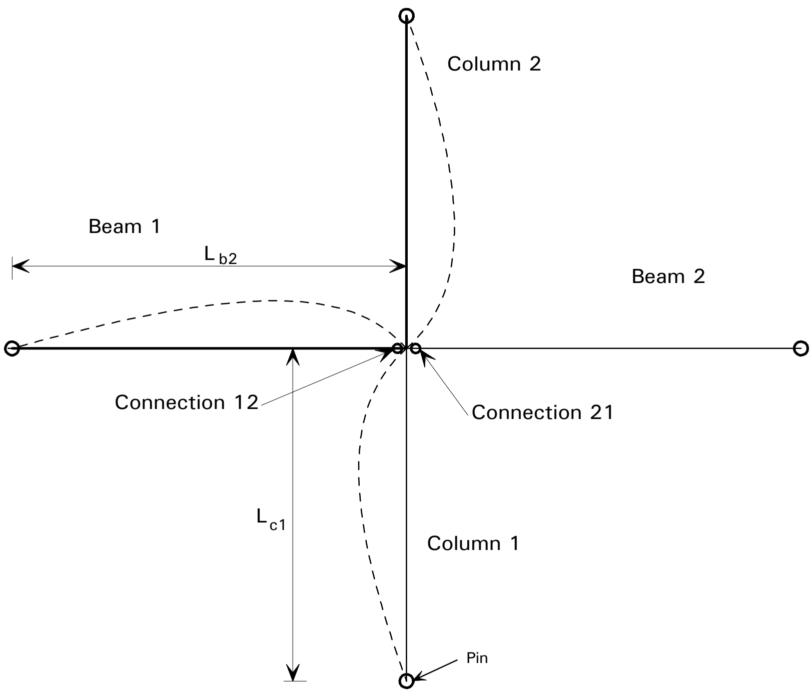


Figure B.4 *Subframe members*

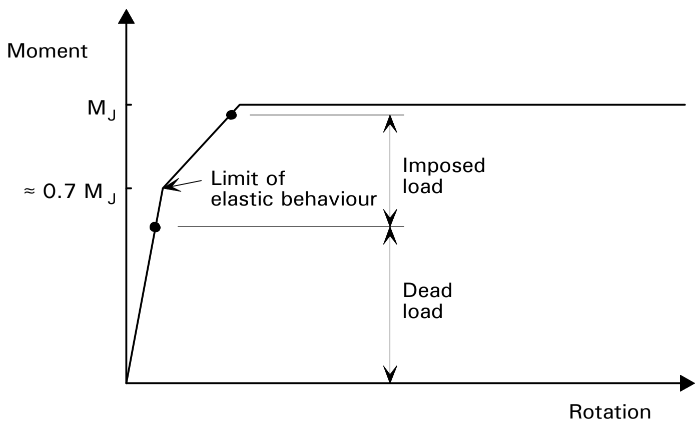


Figure B.5 *Schematic connection moment-rotation response, indicating typical levels of moment under dead and imposed load*

The designer must decide whether or not it is necessary to calculate apparent column stiffness for each individual case. For example, in a symmetric frame it would not be necessary to quantify column stiffness when calculating dead load deflections if equal dead load were present either side of a node, so that no column rotation takes place. The apparent column stiffness can be taken as infinite. However, imposed load might be present on only one side of a node, producing column rotation. Apparent column stiffness must then be quantified, and allowed for in calculations of imposed load deflections.

The stiffness of a pin-ended member with moment applied at one end is given (see Figure B.6) by:

$$k = \frac{M}{q} = \frac{3EI}{L}$$

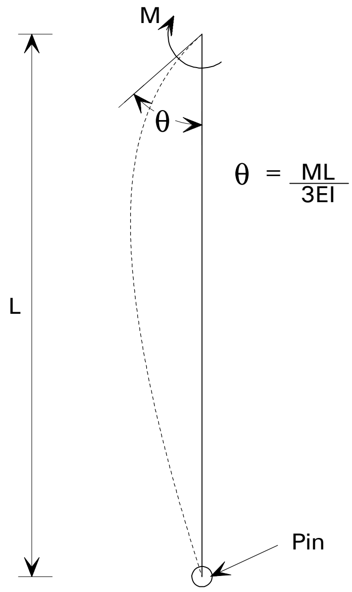


Figure B.6 *End rotation of a pin-ended member subject to moment*

Apparent column stiffness is obtained by summing the stiffnesses of the supporting members[11]; with reference to Figure B.4, for calculating deflections of Beam 2, the supporting members are Column 1, Column 2, and Beam 1. The apparent column stiffness for the left hand support of Beam 2 is therefore given by:

$$k_{\mathrm{c,app}} = k_{\mathrm{c1}} + k_{\mathrm{c2}} + k_{\mathrm{b1}}$$

where:

$$k_{\mathrm{ci}} = \frac{3EI_{\mathrm{ci}}}{L_{\mathrm{ci}}}$$

The stiffness of Beam 1 must allow for the stiffness of the connection with which it is joined to the column. This beam stiffness is given by:

$$k_{b1} = \left[\frac{L_{b1}}{3EI_{b1}} + \frac{1}{k_{j12}}\right]^{-1}$$

All three supporting members are conservatively assumed to be pinned at their extremity.

The **'support' stiffness** (k_s), which represents the stiffness of the connection and all the elements behind it (column, adjacent connection and beam), is therefore given by:

$$k_s = \left[\frac{1}{k_{c,app}} + \frac{1}{k_{j21}}\right]^{-1}$$

Beam stiffness

The effective stiffness of the beam whose deflection is being checked (Beam 2 in Figure B.4) is given by:

$$k_{b2} = \frac{aEI_{b2}}{L_{b2}}$$

The value of α depends on the beam type (internal or external span) and loading. The fact that effective stiffness is dependent on these two parameters, in addition to EI_{b2}/L_{b2}, can be illustrated by considering two cases of simply supported beams:

(i) the deflection of a simply supported beam subject to UDL is given by:

$$d = \frac{5}{384}\frac{wL^4}{EI}$$

(ii) the deflection of the same beam subject to a central point load P, equal to wL, is given by:

$$d = \frac{1}{48}\frac{PL^3}{EI} = \frac{8}{384}\frac{wL^4}{EI}$$

From this illustration it is clear that the magnitude of the beam deflection, which is a measure of its effective stiffness, depends not only on EI/L, but also on the configuration of the applied loading.

Appropriate values of α can be derived using Roark's formulae for stress and strain to define M and θ [(12)]. Use of the design curves given in Figures B.2 and B.3 is compatible with such a derivation (see Section B.3 of this Appendix). Typical examples for internal spans are:

- uniformly distributed load α = 2.0
- point load at mid-span α = 2.0
- point load at third span point α = 2.4 for 'near' end
- point load at third span point α = 1.5 for 'far' end.

A value of α equal to 2.0 can be used for most internal spans. A value of α equal to 3.0 can be used for all external spans. External spans appear to be stiffer than internal spans (α = 3.0 rather than say 2.0) because moment is only applied at one end. For an internal span, end rotation increases by 50% due to the assumed application of an equal and opposite moment at the far end of the beam.

B.3 Derivation of design curve

The design curves given in Figures B.2 and B.3 were derived using elastic analysis software. A range of beam section sizes and spans was analysed under different loading regimes, with various assumed 'support' stiffnesses for each case. Values of end moment were recorded for each case, and compared with built-in end moments to enable plotting as moment redistribution (see Figure B.7).

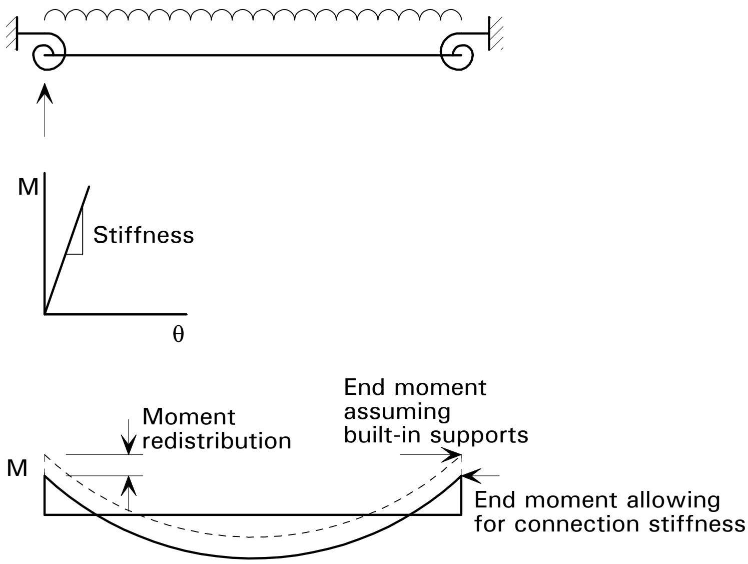


Figure B.7 *Model used for elastic analysis; example with schematic results*

Having calculated moment redistribution for each case, and knowing the 'support' stiffness, the results were plotted after having calculated beam stiffness using the procedures given in Section B.2 of this Appendix. The design curves represent a mean through the plotted values. Provided the designer calculates the 'support' and beam stiffnesses in the prescribed way, he can therefore use the design curves to predict the results of analysis software.

B.4 Validation of procedure

The procedure given in Section B.2 of this Appendix was validated by analysing a complete subframe, comprising beams and columns, using elastic analysis software. Details of the subframe are given in Figure B.8.

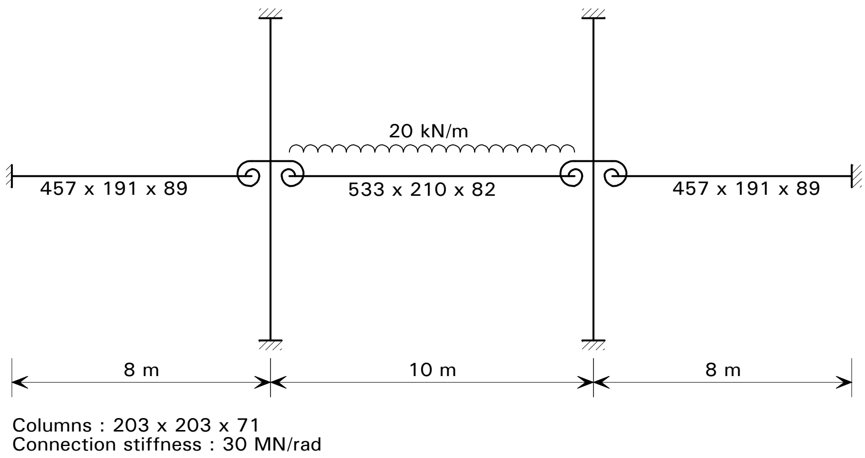


Figure B.8 *Subframe used to validate design procedure*

The software calculated a mid-span deflection for Beam 2 equal to 15.8 mm. The design procedures predicted a deflection of 15.9 mm, confirming their applicability.

APPENDIX C Connection capacity tables

C.1 Notes on use of the tables

The tables given in this Appendix cover connections suitable for use in semi-continuous braced frames, using design procedures described in this document. Capacity tables for connections using M20 8.8 bolts, with flush or extended end plates, are presented first, followed by similar connections with M24 8.8 bolts. A table defining dimensions for detailing is given at the end of the Appendix.

The moment capacity of the connections shown may be used for all weights of beams (within the serial sizes indicated), in grade S275 or S355 steel. All end plates are grade S275. Local column capacities must be checked as described below.

For the connection to work in the intended manner, it is important that plate size and steel grade, minimum bolt and weld sizes, and dimensions between bolt centres etc are strictly adhered to. Any deviations, other than those within normal construction tolerances, may either reduce the capacity of the connection, compromise its ductility or invalidate the column check. A table of dimensions for detailing to suit individual beams is provided at the end of this Appendix.

Axial forces in the beams are ignored in the design method given in this document, and therefore the standard connection capacities are calculated without considering them. These connections should not therefore be used to transmit axial forces as part of a bracing system.

C.1.1 Beam side

Moment Capacity

The moment capacity for the beam side of the connection is calculated using the method given in Reference 6. Bolt row forces are shown in the diagram.

An asterisk * indicates that, with the detail illustrated, the beam sections noted can only be used in grade S355 steel. When grade S275 steel is used the beam compression flange capacity is less than ΣF_r. Although the connection capacity could be reduced to allow for this 'weak link', the adverse effect on ductility cannot be allowed for, and the choice of detail should be revised.

If reduced bolt row forces on the column side (see C.1.2) limit development of the beam side forces shown, a reduced moment capacity must be calculated using these reduced forces.

Dimension A

is the lever arm from the centre of compression to the lowest row of tension bolts.

Weld Sizes

All flange welds should be full strength, with a minimum visible fillet of 10 mm. All web welds should be at least continuous 8 mm fillets.

C.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 corresponding beam force(s). Where the column has a smaller capacity, reduced bolt row forces are shown. A reduced moment may be determined from these lower forces, or the column flange may be stiffened in the tension zone[6].

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

Where tension zone stiffening is employed, the bolt row forces must be re-calculated and the compression zone checked[6].

Compression Zone

A tick ✓ in the table indicates that the column web has a greater compression capacity than the sum of the bolt row forces (ΣF_r). Note that when the column side tension zone governs the bolt forces, the stated adequancy or otherwise of the column compression zone is in relation to these 'reduced' bolt values. The check was made assuming a stiff bearing length from the beam side of the connection of 50 mm, regardless of beam size.

S in the table shows that the column web compression capacity (given in brackets) is lower than the sum of the bolt row forces (ΣF_r). The web must be stiffened to resist ΣF_r.

Panel Shear Capacity

The panel shear capacity is the capacity of the column web. The applied panel shear must take account of beams connecting onto both flanges, and the direction of the applied moments. When the applied moments from two beams are in opposite directions, the panel shear forces from the beams tend to cancel each other out.

C.2 Example of capacity table use

Determine the connection capacity for a detail with two rows of M24 8.8 bolts and a 250 × 15 mm extended endplate, connecting a 686 × 254 mm beam to an internal 254 × 254 × 73 column (page 57):

	Beam Side	Column Side
Moment capacity	358kNm	Tension zone: 2nd bolt row limited to 274 kN Reduced moment capacity = (274 × 0.610) + (242 × (0.610 + 0.10)) = **339kNm** Compression zone: stiffening is required to resist (274 + 242) = 516 kN
Vertical shear	**634kN** (without additional shear bolts)	
Column Web Panel Shear		Opposing beams give zero shear across the column web.

Note: governing values for design are shown in bold

C.3 Standard Connections

Contents

End Plate	Type	Bolt	Tension Bolt Rows	Page
200 × 12	Flush	M20	1	46
200 × 12	Flush	M20	2	47
250 × 12	Flush	M20	2	48
200 × 12	Extended	M20	2	49
250 × 12	Extended	M20	2	50
200 × 12	Extended	M20	3	51
250 × 12	Extended	M20	3	52
200 × 15	Flush	M24	1	53
200 × 15	Flush	M24	2	54
250 × 15	Flush	M24	2	55
200 × 15	Extended	M24	2	56
250 × 15	Extended	M24	2	57
200 × 15	Extended	M24	3	58
250 × 15	Extended	M24	3	59

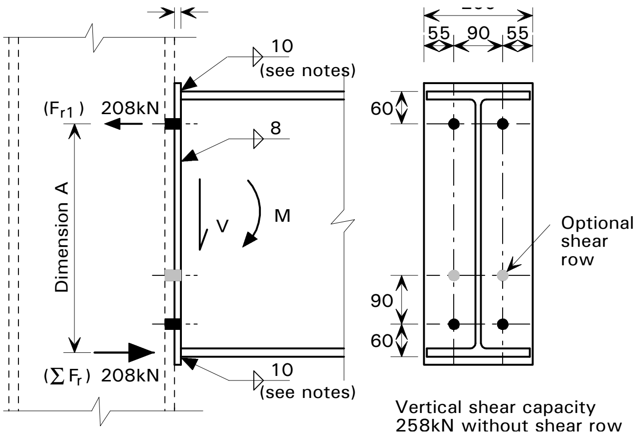
Dimensions for detailing are shown on page 60

BEAM SIDE

1 ROW M20 8.8 BOLTS
200 × 12 S275 FLUSH END PLATE

BEAM - S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
457 × 191	387	**80**
457 × 152	384	**80**
406 × 178	337	**70**
406 × 140	333	**69**
356 × 171	287	**60**
356 × 127	284	**59**
305 × 165	239	**50**
305 × 127	239	**49**
305 × 102	241	**50**
254 × 146	187	**39**
254 × 102	191	**40**



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Compn. Zone	COLUMN Serial Size	S355 Compn. Zone	S355 Tension Zone F_{r1} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	356 × 368 × 202	✓	✓	1300
849	✓	✓	177	✓	✓	1110
725	✓	✓	153	✓	✓	944
605	✓	✓	129	✓	✓	788
1037	✓	✓	305 × 305 ×198	✓	✓	1350
816	✓	✓	158	✓	✓	1060
703	✓	✓	137	✓	✓	916
595	✓	✓	118	✓	✓	775
503	✓	✓	97	✓	✓	649
882	✓	✓	254 × 254 × 167	✓	✓	1150
685	✓	✓	132	✓	✓	893
551	✓	✓	107	✓	✓	718
434	✓	✓	89	✓	✓	566
360	✓	✓	73	✓	✓	465
459	✓	✓	203 × 203 × 86	✓	✓	598
353	✓	✓	71	✓	✓	460
322	✓	✓	60	✓	✓	415
272	✓	✓	52	✓	✓	351
245	198	✓	46	✓	✓	316

Tension Zone:
F_{r1}
✓ Column satisfactory for bolt row tension values shown for the beam side.
xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

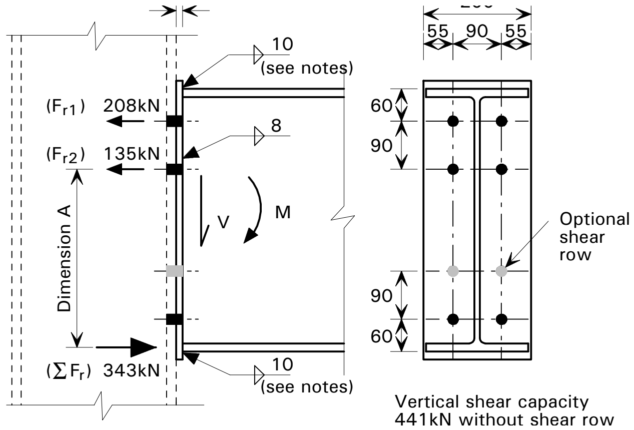
See:
Notes - page 43
Example - page 44

2 ROWS M20 8.8 BOLTS
200 × 12 S275 FLUSH END PLATE

BEAM SIDE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 × 210	372	**150**
457 × 191	297	**123**
457 × 152	294	**122**
406 × 140	247	**105**
406 × 140	243	**102**



Vertical shear capacity 441kN without shear row

COLUMN SIDE

S275				COLUMN Serial Size	S355			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn. Zone		Compn. Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1100
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	✓	✓	89	✓	✓	✓	566
360	✓	✓	✓	73	✓	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	✓	71	✓	✓	✓	460
322	✓	✓	✓	60	✓	✓	✓	415
272	✓	✓	✓	52	✓	✓	✓	351
245	198	97	✓	46	✓	✓	✓	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

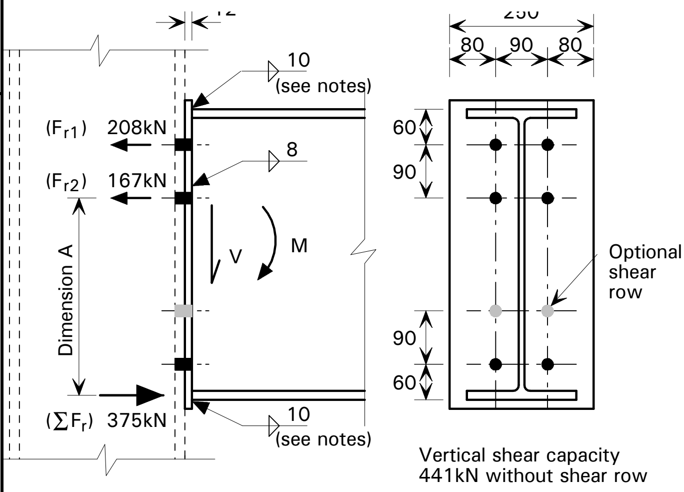
See:
Notes - page 43
Example - page 44

2 ROWS M20 8.8 BOLTS
250 × 12 S275 FLUSH END PLATE

BEAM SIDE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 × 254	520	**220**
610 × 229	445	**190**
533 × 210	372	**160**
457 × 191	297	**131**
457 × 152	294	**129**



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Compn. Zone	COLUMN Serial Size	S355 Compn. Zone	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1302
849	✓	✓	✓	177	✓	✓	✓	1105
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	787
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1062
703	✓	✓	✓	137	✓	✓	✓	915
595	✓	✓	✓	118	✓	✓	✓	774
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1149
685	✓	✓	✓	132	✓	✓	✓	892
551	✓	✓	✓	107	✓	✓	✓	717
434	✓	✓	✓	89	✓	✓	✓	566
360	✓	✓	✓	73	✓	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	✓	71	✓	✓	✓	460
322	✓	✓	✓	60	✓	✓	✓	415
272	✓	✓	S(360)	52	✓	✓	✓	351
245	198	97	✓	46	✓	✓	✓	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity)

See:
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Example - page 44

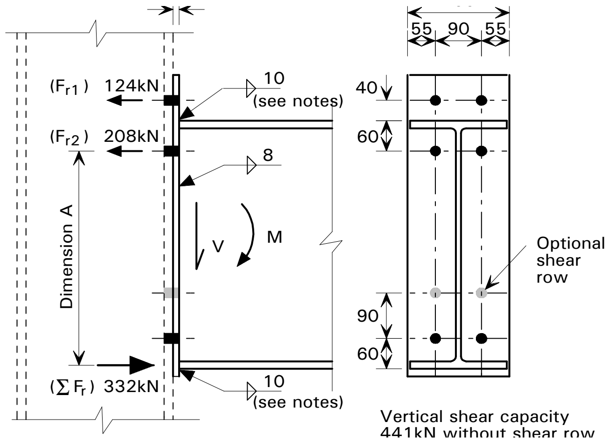
BEAM SIDE

2 ROWS M20 8.8 BOLTS
200 × 12 S275 EXTENDED END PLATE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 × 210	462	**165**
457 × 191	387	**141**
457 × 152	384	**140**
406 × 178	337	**124**
406 × 140	333	**123**
356 × 171	287	**107**
356 × 127	284	**107**
305 × 165	239	**91**
305 × 127	239	**91**
305 × 102*	241	**92**
254 × 146	187	**74**
254 × 102*	191	**75**

* 305 × 102 × 25
254 × 102 × 25
254 × 102 × 22
These sections suitable in S355 only



COLUMN SIDE

S275				COLUMN Serial Size	S355			
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Compn Zone		Compn Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	✓	✓	89	✓	✓	✓	566
360	✓	206	✓	73	✓	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	✓	71	✓	✓	✓	460
322	✓	191	✓	60	✓	✓	202	415
272	✓	181	✓	52	✓	✓	190	351
245	✓	107	✓	46	✓	✓	181	316

Tension Zone:
F_{r1} F_{r2}
✓ ✓ Column satisfactory for bolt row tension values shown for the beam side.
✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:
✓ Column capacity exceeds ΣF_r

See:
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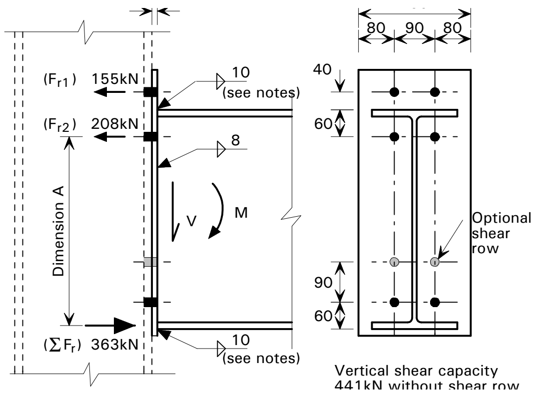
Example - page 44

2 ROWS M20 8.8 BOLTS
250 × 12 S275 EXTENDED END PLATE

BEAM SIDE

BEAM -S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
686 × 254	610	**236**
610 × 229	535	**209**
533 × 210	462	**183**
457 × 191	387	**156**
457 × 152	384	**155**



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Compn. Zone	COLUMN Serial Size	S355 Compn. Zone	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	✓	✓	89	✓	✓	✓	566
360	✓	206	✓	73	✓	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	✓	71	✓	✓	✓	460
322	✓	191	✓	60	✓	✓	202	415
272	✓	181	✓	52	✓	✓	190	351
245	✓	107	✓	46	✓	✓	181	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

See:
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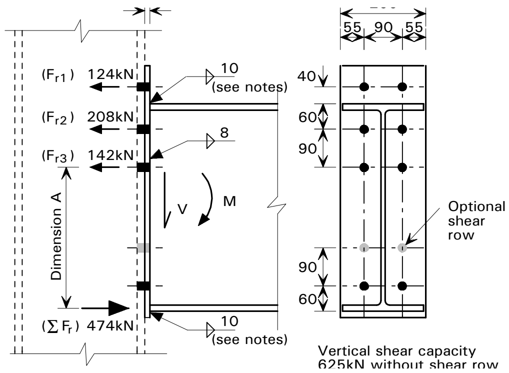
Example - page 44

BEAM SIDE

3 ROWS M20 8.8 BOLTS
200 × 12 S275 EXTENDED END PLATE

BEAM - S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
533 × 210	372	**220**
457 × 191	297	**184**
457 × 152	294	**182**
406 × 178	247	**160**
406 × 140*	243	**155**

*406 × 140 × 39 is suitable in S355 only



COLUMN SIDE

S275						S355				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn Zone	COLUMN Serial Size	Compn Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✓	✓	✓	✓	356 × 368 × 202	✓	✓	✓	✓	1300
849	✓	✓	✓	✓	177	✓	✓	✓	✓	1110
725	✓	✓	✓	✓	153	✓	✓	✓	✓	944
605	✓	✓	✓	✓	129	✓	✓	✓	✓	788
1037	✓	✓	✓	✓	305 × 305 × 198	✓	✓	✓	✓	1350
816	✓	✓	✓	✓	158	✓	✓	✓	✓	1060
703	✓	✓	✓	✓	137	✓	✓	✓	✓	916
595	✓	✓	✓	✓	118	✓	✓	✓	✓	775
503	✓	✓	✓	✓	97	✓	✓	✓	✓	649
882	✓	✓	✓	✓	254 × 254 × 167	✓	✓	✓	✓	1150
685	✓	✓	✓	✓	132	✓	✓	✓	✓	893
551	✓	✓	✓	✓	107	✓	✓	✓	✓	718
434	✓	✓	✓	✓	89	✓	✓	✓	✓	566
360	✓	206	✓	*S (436)*	73	✓	✓	✓	✓	465
459	✓	✓	✓	✓	203 × 203 × 86	✓	✓	✓	✓	598
353	✓	✓	✓	✓	71	✓	✓	✓	✓	460
322	✓	191	✓	*S (440)*	60	✓	✓	202	✓	415
272	✓	181	121	*S (360)*	52	✓	✓	190	✓	351
245	✓	107	90	*S (313)*	46	*S (404)*	✓	181	118	316

Tension Zone:

F_{r1} F_{r2} F_{r3}

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

✓ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity)

See:
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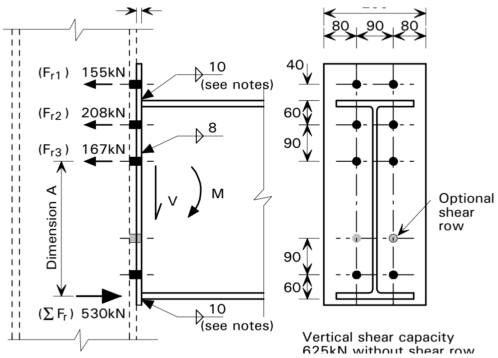
Example - page 44

BEAM SIDE

3 ROWS M20 8.8 BOLTS
250 × 12 S275 EXTENDED END PLATE

BEAM -S275 & S355		
Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 × 254	520	**330**
610 × 229	445	**288**
533 × 210	372	**247**
457 × 191	297	**206**
457 × 152	294	**204**



COLUMN SIDE

S275						S355				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn Zone	COLUMN Serial Size	Compn Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✓	✓	✓	✓	356 × 368 × 202	✓	✓	✓	✓	1300
849	✓	✓	✓	✓	177	✓	✓	✓	✓	1110
725	✓	✓	✓	✓	153	✓	✓	✓	✓	944
605	✓	✓	✓	✓	129	✓	✓	✓	✓	788
1037	✓	✓	✓	✓	305 × 305 × 198	✓	✓	✓	✓	1350
816	✓	✓	✓	✓	158	✓	✓	✓	✓	1060
703	✓	✓	✓	✓	137	✓	✓	✓	✓	916
595	✓	✓	✓	✓	118	✓	✓	✓	✓	775
503	✓	✓	✓	✓	97	✓	✓	✓	✓	649
882	✓	✓	✓	✓	254 × 254 × 167	✓	✓	✓	✓	1150
685	✓	✓	✓	✓	132	✓	✓	✓	✓	893
551	✓	✓	✓	✓	107	✓	✓	✓	✓	718
434	✓	✓	✓	✓	89	✓	✓	✓	✓	566
360	✓	206	✓	*S (436)*	73	✓	✓	✓	✓	465
459	✓	✓	✓	✓	203 × 203 × 86	✓	✓	✓	✓	598
353	✓	✓	✓	*S (512)*	71	✓	✓	✓	✓	460
322	✓	191	✓	*S (440)*	60	✓	✓	202	✓	415
272	✓	181	121	*S (360)*	52	*S (464)*	✓	190	✓	351
245	✓	107	90	*S (313)*	46	*S (404)*	✓	181	118	316

Tension Zone:

F_{r1} F_{r2} F_{r3}

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

✓ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity)

See:
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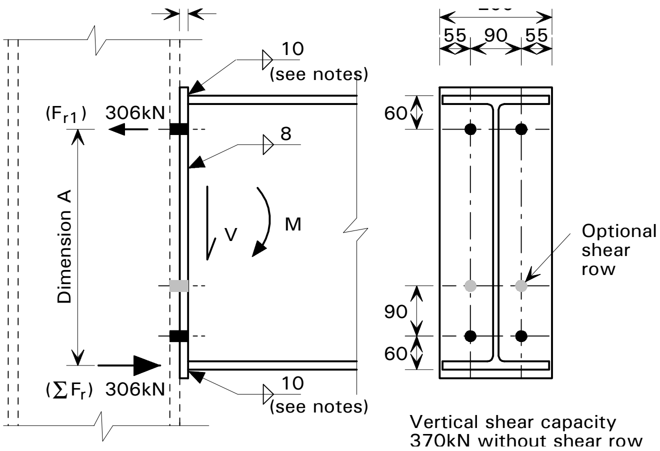
1 ROW M24 8.8 BOLTS
200 × 15 S275 FLUSH END PLATE

BEAM SIDE

BEAM -S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
457 × 191	387	**119**
457 × 152	384	**118**
406 × 178	337	**103**
406 × 140	333	**102**
356 × 171	287	**88**
356 × 127	284	**87**
305 × 165	239	**73**
305 × 127	238	**73**
305 × 102*	241	**74**
254 × 146	187	**57**
254 × 102*	191	**58**

*305 × 102 × 25
254 × 102 × 22

These sections suitable in S355 only



COLUMN SIDE

S275				S355		
Panel Shear Capacity (kN)	**Tension Zone F_{r1} (kN)**	**Compn Zone**	**COLUMN Serial Size**	**Compn Zone**	**Tension Zone F_{r1} (kN)**	**Panel Shear Capacity (kN)**
1000	✓	✓	356 × 368 × 202	✓	✓	1300
849	✓	✓	177	✓	✓	1110
725	✓	✓	153	✓	✓	944
605	✓	✓	129	✓	✓	788
1037	✓	✓	305 × 305 × 198	✓	✓	1350
816	✓	✓	158	✓	✓	1060
703	✓	✓	137	✓	✓	916
595	✓	✓	118	✓	✓	775
503	✓	✓	97	✓	✓	649
882	✓	✓	254 × 254 × 167	✓	✓	1150
685	✓	✓	132	✓	✓	893
551	✓	✓	107	✓	✓	718
434	✓	✓	89	✓	✓	566
360	297	✓	73	✓	✓	465
459	✓	✓	203 × 203 × 86	✓	✓	598
353	✓	✓	71	✓	✓	460
322	297	✓	60	✓	✓	415
272	265	✓	52	✓	296	351
245	204	✓	46	✓	263	316

Tension Zone:

F_{r1}

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

xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✓ Column capacity exceeds ΣF_r

See:
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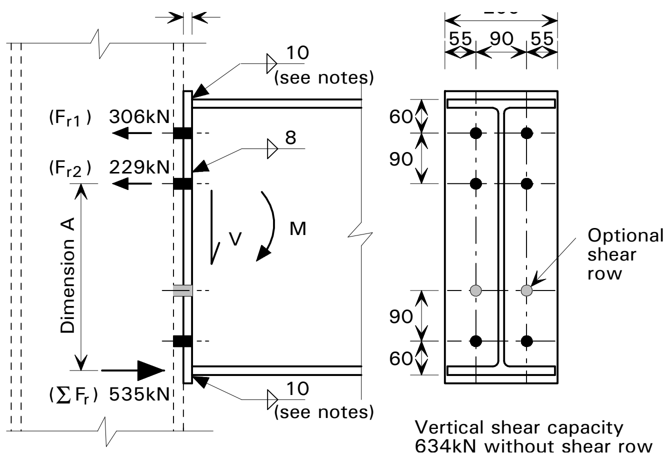
BEAM SIDE

2 ROWS M24 8.8 BOLTS
200 × 15 S275 FLUSH END PLATE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 × 210	372	**233**
457 × 191	297	**191**
457 × 152	294	**186**
406 × 178	247	**161**
406 × 140*	243	**158**

*406 × 140 × 39 is suitable in S355 only



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Compn Zone	COLUMN Serial Size	S355 Compn Zone	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	✓	✓	89	✓	✓	✓	566
360	297	✓	S *(436)*	73	✓	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	S *(512)*	71	✓	✓	✓	460
322	297	204	S *(440)*	60	✓	✓	✓	415
272	265	118	S *(360)*	52	S *(464)*	296	198	351
245	204	90	✓	46	✓	263	116	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S *(xxx)* Column reduced stiffening to resist ΣF_r (Value is the column web capacity.)

See:
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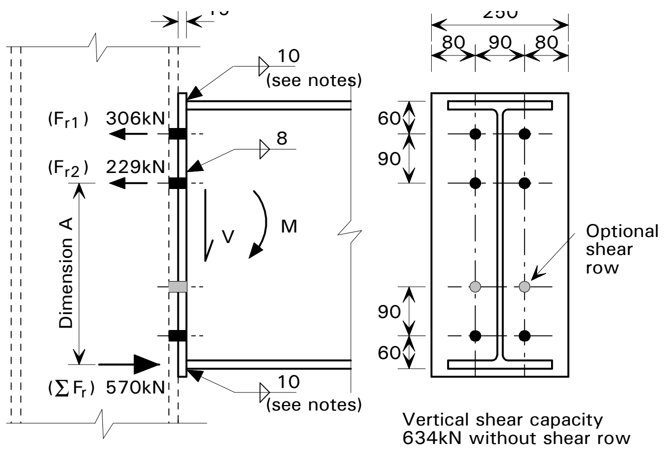
Example - page 44

BEAM SIDE

2 ROWS M24 8.8 BOLTS
250 × 15 S275 FLUSH END PLATE

BEAM -S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
686 × 254	520	**326**
610 × 229	445	**283**
533 × 210	372	**240**
457 × 191	297	**197**
457 × 152	294	**195**



COLUMN SIDE

S275					S355			
Panel Shear Capacity (kN)	**Tension Zone** F_{r1} (kN)	**Tension Zone** F_{r2} (kN)	**Compn Zone**	**COLUMN Serial Size**	**Compn Zone**	**Tension Zone** F_{r1} (kN)	**Tension Zone** F_{r2} (kN)	**Panel Shear Capacity (kN)**
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	S *(553)*	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	✓	S *(557)*	89	✓	✓	✓	566
360	297	✓	S *(436)*	73	S *(563)*	✓	✓	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	✓	S *(512)*	71	✓	✓	✓	460
322	297	204	S *(440)*	60	S *(568)*	✓	✓	415
272	265	118	S *(360)*	52	S *(464)*	296	198	351
245	204	90	✓	46	✓	263	116	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column reduced stiffening to resist ΣF_r (Value is the column web capacity.)

See:
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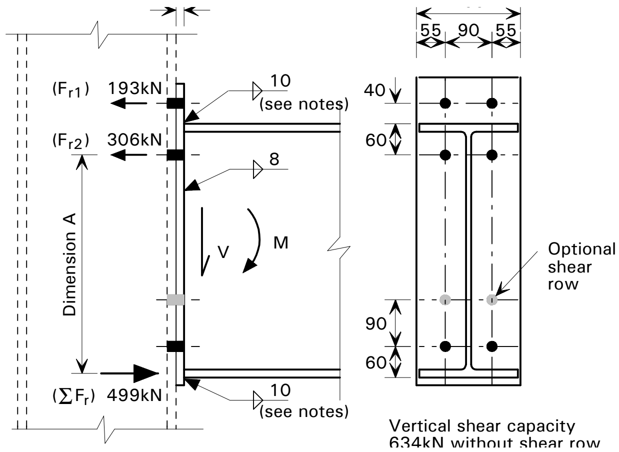
Example - page 44

BEAM SIDE

2 ROWS M24 8.8 BOLTS
200 × 15 S275 EXTENDED END PLATE

BEAM -S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
533 × 210	462	**250**
457 × 191	387	**213**
457 × 152	384	**211**
406 × 178	337	**188**
406 × 140*	333	**186**
356 × 171	287	**163**
356 × 127*	284	**161**
305 × 165	239	**139**
305 × 127	238	**139**
*406 × 140 × 39 356 × 127 × 33		These sections suitable in S355 only



COLUMN SIDE

S275					S355			
Panel Shear Capacity (kN)	**Tension Zone F_{r1} (kN)**	**Tension Zone F_{r2} (kN)**	**Compn Zone**	**COLUMN Serial Size**	**Compn Zone**	**Tension Zone F_{r1} (kN)**	**Tension Zone F_{r2} (kN)**	**Panel Shear Capacity (kN)**
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	301	✓	89	✓	✓	✓	566
360	✓	274	S *(436)*	73	✓	✓	289	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	276	✓	71	✓	✓	293	460
322	✓	221	✓	60	✓	✓	269	415
272	✓	131	✓	52	✓	✓	215	351
245	✓	100	✓	46	✓	✓	129	316

Tension Zone:

F_{r1} F_{r2}

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

✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column reduced stiffening to resist ΣF_r (Value is the column web capacity.)

See:
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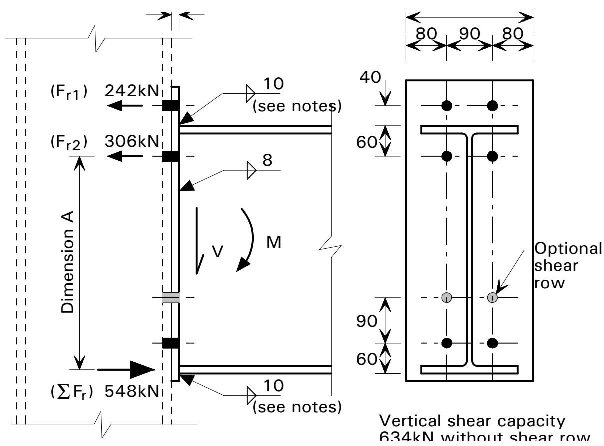
Example - page 44

BEAM SIDE

2 ROWS M24 8.8 BOLTS
250 × 15 S275 EXTENDED END PLATE

BEAM -S275 & S355		
Beam Serial Size	**Dimension 'A' (mm)**	**Moment Capacity (kNm)**
686 × 254	610	**358**
610 × 229	535	**317**
533 × 210	462	**277**
457 × 191	387	**236**



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Compn Zone	COLUMN Serial Size	S355 Compn Zone	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	✓	356 × 368 × 202	✓	✓	✓	1300
849	✓	✓	✓	177	✓	✓	✓	1110
725	✓	✓	✓	153	✓	✓	✓	944
605	✓	✓	✓	129	✓	✓	✓	788
1037	✓	✓	✓	305 × 305 × 198	✓	✓	✓	1350
816	✓	✓	✓	158	✓	✓	✓	1060
703	✓	✓	✓	137	✓	✓	✓	916
595	✓	✓	✓	118	✓	✓	✓	775
503	✓	✓	✓	97	✓	✓	✓	649
882	✓	✓	✓	254 × 254 × 167	✓	✓	✓	1150
685	✓	✓	✓	132	✓	✓	✓	893
551	✓	✓	✓	107	✓	✓	✓	718
434	✓	301	✓	89	✓	✓	✓	566
360	✓	274	S (*436*)	73	✓	✓	289	465
459	✓	✓	✓	203 × 203 × 86	✓	✓	✓	598
353	✓	276	S (*512*)	71	✓	✓	293	460
322	✓	221	S (*440*)	60	✓	✓	269	415
272	✓	131	S (*360*)	52	✓	✓	215	351
245	204	100	✓	46	✓	✓	129	316

Tension Zone:

F_{r1} F_{r2}
✓ ✓ Column satisfactory for bolt row tension values shown for the beam side.
✓ xxx Calculate reduced moment capacity using the reduced bolt row value.

Compression Zone:

✓ Column capacity exceeds ΣF_r
S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity.)

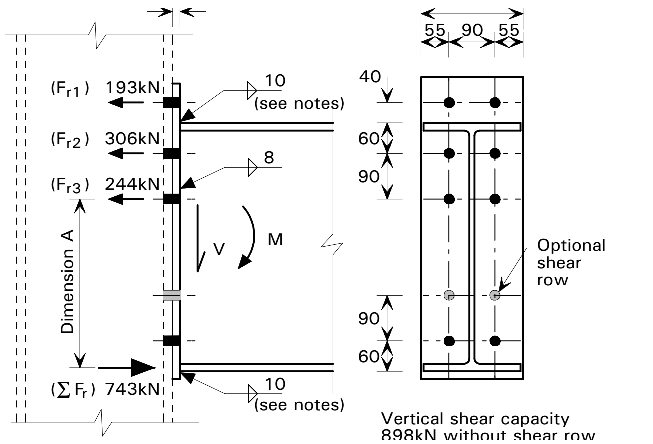
See:
Notes - page 43
Example - page 44

BEAM SIDE

3 ROWS M24 8.8 BOLTS
200 × 15 S275 EXTENDED END PLATE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
533 × 210	372	**342**
457 × 191	297	**286**



COLUMN SIDE

S275					COLUMN Serial Size	S355				
Panel Shear Capacity (kN)	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Compn Zone		Compn Zone	Tension Zone F_{r1} (kN)	Tension Zone F_{r2} (kN)	Tension Zone F_{r3} (kN)	Panel Shear Capacity (kN)
1000	✓	✓	✓	✓	356 × 368 × 202	✓	✓	✓	✓	1300
849	✓	✓	✓	✓	177	✓	✓	✓	✓	1110
725	✓	✓	✓	✓	153	✓	✓	✓	✓	944
605	✓	✓	✓	S *(605)*	129	✓	✓	✓	✓	788
1037	✓	✓	✓	✓	305 × 305 × 198	✓	✓	✓	✓	1350
816	✓	✓	✓	✓	158	✓	✓	✓	✓	1060
703	✓	✓	✓	✓	137	✓	✓	✓	✓	916
595	✓	✓	✓	S *(692)*	118	✓	✓	✓	✓	775
503	✓	✓	✓	S *(553)*	97	S *(713)*	✓	✓	✓	649
882	✓	✓	✓	✓	254 × 254 × 167	✓	✓	✓	✓	1150
685	✓	✓	✓	✓	132	✓	✓	✓	✓	893
551	✓	✓	✓	✓	107	✓	✓	✓	✓	718
434	✓	301	✓	S *(557)*	89	S *(725)*	✓	✓	✓	566
360	✓	274	✓	S *(436)*	73	S *(563)*	✓	289	✓	465
459	✓	✓	✓	S *(701)*	203 × 203 × 86	✓	✓	✓	✓	598
353	✓	276	✓	S *(512)*	71	S *(666)*	✓	293	✓	460
322	✓	221	✓	S *(440)*	60	S *(568)*	✓	269	✓	415
272	✓	131	118	S *(360)*	52	S *(464)*	✓	215	152	351
245	✓	100	90	S *(313)*	46	S *(404)*	✓	129	116	316

Tension Zone:

F_{r1} F_{r2} F_{r3}

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

✓ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

✓ Column capacity exceeds ΣF_r

S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity.)

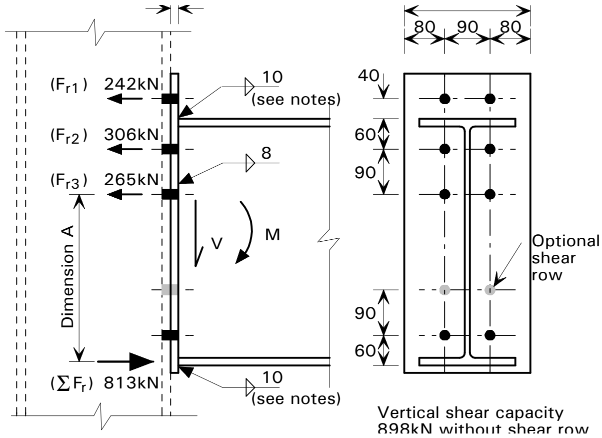
See:
Notes - page 43
Example - page 44

BEAM SIDE

3 ROWS M24 8.8 BOLTS
250 × 15 S275 EXTENDED END PLATE

BEAM -S275 & S355

Beam Serial Size	Dimension 'A' (mm)	Moment Capacity (kNm)
686 × 254	520	**498**
610 × 229	445	**436**
533 × 210	372	**376**
457 × 191	297	**315**



COLUMN SIDE

S275 Panel Shear Capacity (kN)	S275 Tension Zone F_{r1} (kN)	S275 Tension Zone F_{r2} (kN)	S275 Tension Zone F_{r3} (kN)	S275 Compn Zone	COLUMN Serial Size	S355 Compn Zone	S355 Tension Zone F_{r1} (kN)	S355 Tension Zone F_{r2} (kN)	S355 Tension Zone F_{r3} (kN)	S355 Panel Shear Capacity (kN)
1000	✓	✓	✓	✓	356 × 368 × 202	✓	✓	✓	✓	1300
849	✓	✓	✓	✓	177	✓	✓	✓	✓	1110
725	✓	✓	✓	S *(766)*	153	✓	✓	✓	✓	944
605	✓	✓	✓	S *(605)*	129	S *(788)*	✓	✓	✓	788
1037	✓	✓	✓	✓	305 × 305 × 198	✓	✓	✓	✓	1350
816	✓	✓	✓	✓	158	✓	✓	✓	✓	1060
703	✓	✓	✓	✓	137	✓	✓	✓	✓	916
595	✓	✓	✓	S *(692)*	118	✓	✓	✓	✓	775
503	✓	✓	✓	S *(553)*	97	S *(713)*	✓	✓	✓	649
882	✓	✓	✓	✓	254 × 254 × 167	✓	✓	✓	✓	1150
685	✓	✓	✓	✓	132	✓	✓	✓	✓	893
551	✓	✓	✓	S *(744)*	107	✓	✓	✓	✓	718
434	✓	301	✓	S *(557)*	89	S *(725)*	✓	✓	✓	566
360	✓	274	182	S *(436)*	73	S *(563)*	✓	289	✓	465
459	✓	✓	✓	S *(701)*	203 × 203 × 86	✓	✓	✓	✓	598
353	✓	276	✓	S *(512)*	71	S *(666)*	✓	293	✓	460
322	✓	221	155	S *(440)*	60	S *(568)*	✓	269	264	415
272	✓	131	118	S *(360)*	52	S *(464)*	✓	215	152	351
245	204	100	90	S *(313)*	46	S *(404)*	✓	129	116	316

Tension Zone:

F_{r1} F_{r2} F_{r3}

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

✓ xxx xxx Calculate reduced moment capacity using the reduced bolt row values.

Compression Zone:

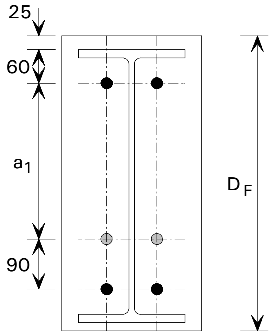
✓ Column capacity exceeds ΣF_r

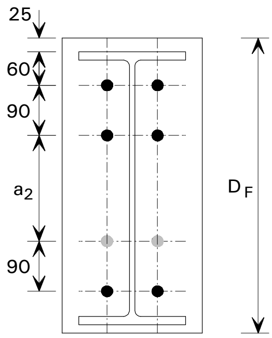
S (xxx) Column requires stiffening to resist ΣF_r (Value is the column web capacity.)

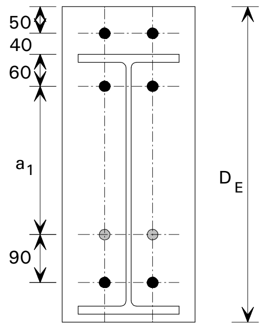
See:
Notes - page 43
Example - page 44

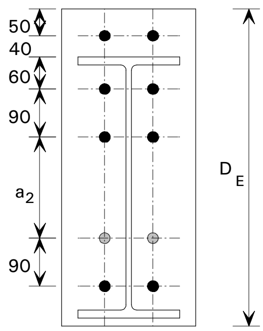
STANDARD CONNECTIONS - DIMENSIONS FOR DETAILING

	dimension a_1 mm	dimension a_2 mm	Flush End Plate Overall Depth D_F mm	Extended End Plate Overall Depth D_E mm
686 × 254 × 170	485	395	750	815
152	480	390		
140	475	385		
125	470	380		
610 × 229 × 140	410	320	670	735
125	400	310		
113	400	310		
101	390	300		
533 × 210 × 122	335	245	600	665
109	330	240		
101	325	235		
92	325	235		
82	320	230		
457 × 191 × 98	260	170	520	585
89	255	165		
82	250	160		
74	250	160		
67	245	155		
457 × 152 × 82	255	165	520	585
74	250	160		
67	250	160		
60	245	155		
52	240	150		
406 × 178 × 74	205	115	470	535
67	200	110		
60	195	105		
54	195	105		
406 × 140 × 46	190	100	450	515
39	185	95		
356 × 171 × 67	155		420	485
57	150			
51	145			
45	140			
356 × 127 × 39	145		410	475
33	140			
305 × 165 × 54	100		360	425
46	95			
40	95			
305 × 127 × 48	100		360	425
42	95			
37	95			
305 × 102 × 33	105		370	435
28	100			
25	95			
254 × 146 × 43	50		310	375
37	45			
31	45			
254 × 102 × 28	50		310	375
25	45			
22	45			









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