SCI PUBLICATION 263

Wind-moment Design of Low Rise Frames

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Published by:

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Publication Number: SCI-P-263

ISBN 1 85942 097 4

British Library Cataloguing-in-Publication Data.
A catalogue record for this book is available from the British Library.

FOREWORD

This publication has been prepared by Mr Paul Salter and Dr Graham Couchman of The Steel Construction Institute and Professor David Anderson of the University of Warwick. The analytical work leading to the publication was carried out by Dr N Brown and Dr M Md Tahir of the University of Warwick. Valuable comments were received during the drafting from Dr R M Lawson of The Steel Construction Institute.

Current SCI publications related to the wind-moment method are:

Wind-moment design for unbraced frames (SCI P082)

Joints in steel construction: Moment connections (SCI P207, published jointly with BCSA)

Wind-moment design of unbraced composite frames (SCI P264, to be published in 2000).

This publication is effectively a replacement for publication SCI P082, which was first published in 1991. It reflects the results of more recent studies, for instance considering frames that are unbraced in both principal directions. It is limited to low rise frames in recognition of the fact that wind-moment design, although possible, is not recommended for building frames in excess of four storeys.

Part-funding from the Department of the Environment, Transport and the Regions under the Partners in Innovation initiative is gratefully acknowledged, as is additional funding received from Corus (formerly British Steel) Sections, Plates & Commercial Steels.

CONTENTS

SUMMARY

This publication presents procedures for the design of *wind-moment* frames in accordance with BS 5950-1. In this method of design, the frame is made statically determinate by treating the connections as *pinned* under vertical loads and *fixed* under horizontal loads (with certain assumed points of zero moment). The publication gives design procedures for frames that are braced in the minor axis direction and for frames that do not have bracing in either principal direction. The limitations of the method, which differ slightly for these two cases, are explained. In particular, it should be noted that the method is only recommended for low-rise frames up to four storeys high.

In addition to design procedures for the ultimate and serviceability limit states, fully worked design examples are presented for two cases. The publication also reproduces the resistance tables for standard *wind-moment* connections taken from SCI/BCSA publication P207 *Joints in steel construction: Moment connections*. These connections use flush or extended end plates and grade 8.8 M20 or M24 bolts, and achieve sufficient rotation capacity by ensuring that the moment resistance is not governed by bolt or weld failure.

'Wind-moment design' de portiques de faibles hauteurs

Resumé

La publication présente des procédures pour le 'wind-moment design' de portiques qui conforme à la norme BS 5950-1. Dans cette méthode de dimensionnement, le portique est rendu isostatique en traitant les assemblages comme des rotules sous charges verticales et comme des encastrements sous charges horizontales (avec certains points supposés à moment nul). La publication donne des procédures de dimensionnement pour des portiques contreventés dans la direction de l'axe faible et pour des portiques sans contreventement. Les limitations de la méthode, qui sont peu différentes dans ces deux cas, sont expliquées. En particulier, il doit être mentionné que la méthode n'est recommandée que pour des cadres jusqu'à quatre niveaux.

Des procédures de dimensionnement à l'état ultime et en service sone expliquées, et deux exemples complets sont présentés. La publication donne des tables de résistance pour des assemblages standardisés qui sont reprises de la publication P207 Assemblages de constructions métalliques: assemblages rigides *du SCI/ BCSA. Ces assemblages utilisent des plaques d'abouts courtes ou étendues et des boulons de nuance 8.8, M20 ou M24 et fournissent une capacité de rotation suffisante pour s'assurer que la résistance aux moments de flexion n'est pas conditionnée par la rupture d'un boulon ou d'une soudure.*

‘Wind-Moment-Berechnung’ von Rahmen geringer Höhe

Zusammenfassung

Diese Publikation präsentiert Vorgehensweisen für die Berechnung von Rahmen unter Einwirkung von Momenten infolge Windlasten (“wind-moment frames”) nach BS 5950-1. Bei dieser Methode wird das Tragwerk statisch bestimmt gemacht durch Annahme von gelenkigen Verbindungen unter vertikalen Lasten und biegesteifer Verbindungen unter horizontalen Lasten (mit gewissen angenommenen Momenten-Nullpunkten). Die Publikation zeigt Berechnungsweisen auf für Rahmen die bezüglich der schwachen Achse unverschieblich sind und für Rahmen die verschieblich sind. Die Grenzen der Methode, die sich für die beiden Fälle leicht unterscheiden, werden erläutert. Besonders sollte beachtet werden, daß die Methode nur für Rahmen geringer Höhe mit bis zu vier Geschossen empfohlen wird.

Zusätzlich zu den Berechnungsmethoden im Grenzzustand der Tragfähigkeit und Gebrauchstauglich-keit werden Berechnungsbeispiele für die beiden Fälle vorgestellt. Die Veröffentlichung reproduziert auch die Tabellen für Standard-Verbindungen aus der SCI/BCSA-Publikation P207 Verbindungen im Stahlbau: Momenten-Verbindungen. *Diese Verbindungen haben bündige oder überstehende Stirnplatten mit Schrauben M20 oder M24 der Güte 8.8 und weisen ausreichende Rotationskapazität auf, ohne Schrauben- oder Schweißnahtversagen.*

Progettazione per azioni orizzontali di telai in acciaio con modesto numero di piani

Sommario

Questa pubblicazione presenta le procedure per la progettazione di telai resistenti alle azioni orizzontali in accordo alla normativa BS 5950-parte 1. Sulla base di questo approccio progettuale, il telaio viene considerato isostatico con connessioni trave-colonna modellate a cerniera se soggetto alle azioni verticali, mentre, in presenza di forze orizzontali i nodi sono considerati rigidi (ipotizzando la presenza di cerniere localizzate in predeterminate zone della struttura). La pubblicazione fornisce le procedure di progetto sia per quei sistemi intelaiati che sono controventati nella direzione di minore rigidezza sia per quelli che non sono dotati di controventi nelle due direzioni principali. Per entrambe le tipologie strutturali sono definite e trattate nel dettaglio le differenti limitazioni del metodo. In particolare, si sottolinea che il metodo è applicabile soltanto per strutture di modesta altezza e con un massimo di quattro piani.

In aggiunta alle procedure di progetto relative agli stati limite sia di servizio sia ultimi, sono presentati alcuni esempi completi per le due tipologie strutturali in esame. La pubblicazione riporta anche le tabelle di resistenza per i collegamenti più comuni nei telai resistenti alle azioni orizzontali. Tali collegamenti, che sono considerati anche nella pubblicazione SCI/BCSA numero P207 Giunti in telai in acciaio: giunti in grado di trasferire azione flettente*, sono realizzati con piatti saldati all'estremità della trave e bullonati alla colonna sia in spessore si trave, sia estesi oltre l'ingombro della trave. Viene fatto riferimento a bullonature realizzate con bulloni dal diametro di 20mm e 24mm di acciaio con classe di resistenza 8.8. Tali connessioni sono in grado di garantire una capacità rotazionale sufficiente affinché la resistenza del nodo all'azione flettente non sia governata dalle rotture dei bulloni o delle saldature.*

Proyecto de pórticos bajos ante carga de viento

Resumen

Esta publicación presenta métodos para el proyecto de pórticos contraviento de acuerdo con BS 5950-1. En este método de proyecto. el pórtico se considera isostático al considerar articulación de las uniones ante cargas verticales y rígidas ante cargas horizontales (con hipótesis adicionales sobre la situación de los puntos de momento nulo). La publicación da métodos de proyecto tanto para pórticos arriostrados en la dirección del eje menor como para los que no tienen arriostramientos en ninguna dirección principal. Se explican las limitaciones del método que difieren ligeramente en ambos casos. Debe observarse en particular que el método solo es recomendable para pórticos bajos de hasta cuatro plantas.

Además de los métodos de proyecto para los estados límites de servicio y último se presentan ejemplos totalmente desarrollados en dos casos. La publicación también reproduce las tablas de existencia para uniones contraviento típicas tomadas de la publicación P207 de la SCI/BCSA Uniones en estructuras de acero: Uniones rígidas. *Estas uniones usan suficiente capacidad de rotación controlando que la resistencia a flexión no esté controlada por la rotura de los pernos o de las soldaduras.*

Dimensionering av ramverk mot vindlaster

Sammanfattning

Denna publikation presenterar metoder för dimensionering av ramar mot vindlaster enligt BS 5950-1. I dimensioneringsmetoden är ramen statiskt bestämd genom att knutpunkterna behandlas såsom ledat infästa vid vertikala laster och fast inspända vid horisontella laster (under antagande av vissa punkter med noll-moment). Publikationen ger dimensioneringsmetoder för ramar som är stagade i den veka axelns riktning och för ramar som inte har någon stagning i någondera av huvudriktningarna. Metodens begränsningar, vilka är något olika för de två fallen, förklaras. Speciellt bör det noteras att metoden endast rekommenderas för ramverk upp till fyra våningars höjd.

Utöver dimensioneringsmetoder för brott- och bruksgränsstadiet, redovisas även två helt genomarbetade exempel. Publikationen återger även hållfasthetstabellerna för standardiserade vindlastupptagande knutpunkter vilka finns i SCI/BCSA publikation P207 Joints in steel constructions: Moment connections. *Dessa knutpunkter har icke utskjutande eller utskjutande ändplattor och M20 eller M24 bultar i hållfasthetsklass 8.8, och erhåller tillräcklig rotationskapacitet genom att tillse att böjmomentkapaciteten inte avgörs av bult- eller svetsbrott.*

NOTATION

A_g	gross cross-sectional area of member
F_c	axial compression due to applied loads
h	storey height
H	horizontal force per bay
L	distance between levels at which both axes of the column section are restrained or, alternatively, the beam span
L_E	effective length of member
m	equivalent uniform moment factor
m	internal moment
M_b	buckling resistance moment
M_{bs}	buckling resistance moment for columns in simple multi-storey construction
M_x	applied end moment about the major axis
M_y	applied end moment about the minor axis
P_c	compressive resistance of member
p_y	design strength of steel
p_c	compression strength of column
r_y	radius of gyration about the minor axis
S	shear force in column
V	shear force in beam
W	horizontal load applied to frame
Z_y	elastic section modulus about the minor axis
8_{LT}	equivalent slenderness of beam
8	slenderness of column

1 INTRODUCTION

When a steel frame is unbraced, it is usual to rely on the bending resistance and stiffness of the connections to resist wind forces. A simple design method, termed the *wind-moment* or *wind-connection* method, may be used to design such frames. The assumptions made in this method render the structure statically determinate and allow the structure to be analysed using simple manual techniques.

The design method proposed in this publication applies to low rise frames of four storeys or less and assumes that:

- under vertical loads the connections act as simple nominally pinned connections (see Figure 1.1)
- under wind loads the connections behave as nominally rigid joints. Points of contraflexure are assumed to occur at the mid-height of the columns and the mid-length of the beams (see Figure 1.2).

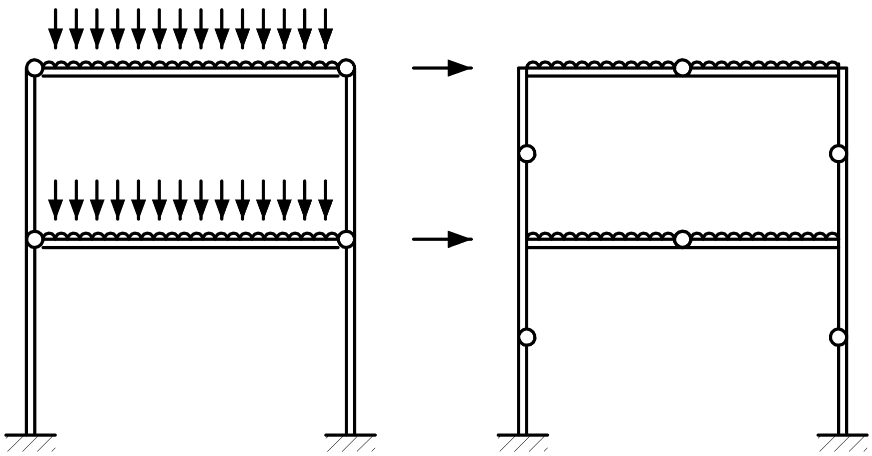

Figure 1.1 *Frame assumptions for the wind-moment method*

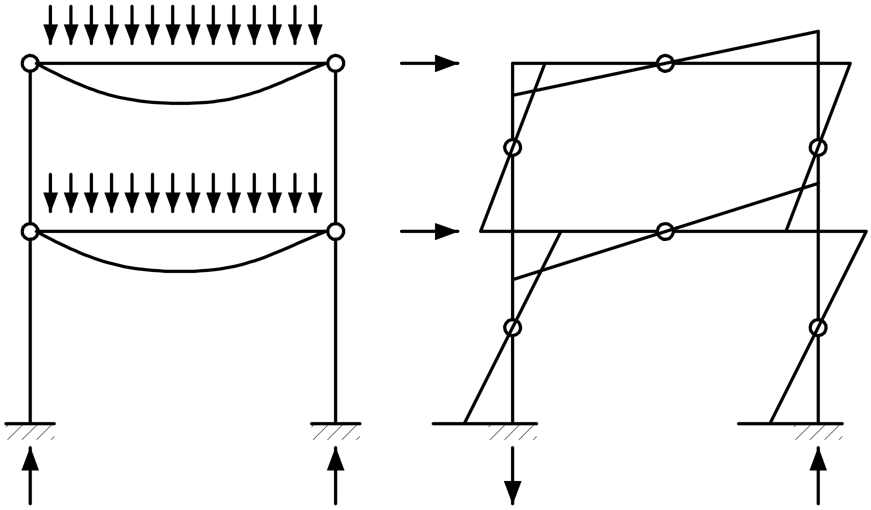

Figure 1.2 *Internal moments and forces according to the wind-moment method*

Procedures are given in this publication for the design of:

C frames that are braced to prevent minor axis sway of the columns at each roof level and each floor level

C frames that contain no sway bracing in either principal direction (subject to additional limits of application).

The first step in the design sequence is to design the beams for the ultimate limit state (ULS) fully factored vertical loads, assuming a nominal end fixity moment of 10%. The frame is then analysed under wind loads, with the assumption that the beam-to-column connections behave in a rigid manner. The internal forces and moments are then combined using the principle of superposition, and adopting appropriate load factors for each combination. Design for the ULS is completed by amending the initial section sizes and connection details, when necessary, so that they can withstand the combined effects.

Connections are required to resist the moments due to wind loading as well as the shear forces due to vertical loading. In reality, the connections may also attract significant moments due to the gravity loading, and they should have sufficient rotation capacity to be able to redistribute these extra moments from the connections into the spans. However, the ductility of the connections need not be a direct concern of the designer provided that standard connections with proven rotation capacity are used. Typical connections with sufficient ductility are given in the BCSA/SCI publication *Joints in steel construction: Moment connections*[1]. Capacity tables for these standard *wind-moment* connections given in that publication are reproduced in Appendix D of this publication.

Second order effects due to frame sway, often referred to as *P-)* effects, are allowed for by using effective lengths for the columns that are greater than the lengths between floor levels. The need for complicated second order calculations is therefore avoided. The additional moments generated due to sway can be significant and for this reason the method is not recommended for high-rise buildings.

When checking the serviceability limit state, lateral deflections due to wind loading can be calculated using a manual method, as explained in the worked example.

1.1 Benefits of *wind-moment* design

The main advantages of this method from a designer's point of view are its simplicity and its suitability for manual calculations. The method is based on procedures familiar to those designing nominally pinned and braced structures on a regular basis. As the frame is assumed to be statically determinate, internal moments and forces are not dependent on the relative stiffnesses of the members. There is therefore no need for any iteration to redetermine moments and forces as member sizes are refined.

The major advantage of *wind-moment* frames from a construction viewpoint is the relative simplicity of the steelwork when compared with fully rigid construction. Much of the work carried out by steelwork contractors is concerned with making the connections, and it has been estimated that the fabrication and workshop handling costs associated with the connections can be as high as 50% of the total cost of the erected steelwork[2]. Any method that allows the connections to be simplified and therefore reduces fabrication input, can significantly reduce the cost of the erected steel frame. The type of connections that are adopted in *wind-moment* frames are simpler than those required for fully rigid construction, particularly for major axis connections.

1.2 Connections

As noted previously, the connections are assumed to be nominally pinned under vertical loading but are designed to resist moments when the frame is subject to lateral wind forces. It is therefore an essential requirement of the *wind-moment* method that the connections should be sufficiently ductile to accommodate large rotations. End plate connections can achieve sufficient ductility, provided that bolt or weld failure is avoided by appropriate detailing. In carrying out the background studies to justify the method given in this design guide, it has been assumed that the connections adopted will comply with the performance of the standard wind-moment connections detailed in *Joints in steel construction: Moment connections* [1] and repeated in Appendix D of this publication. These connections are either flush or extended end plate details that generally require little or no stiffening of the column. A typical detail is illustrated in Figure 1.3. Wind-moment connections should be symmetric in such a way as to provide the same moment resistance in both hogging and sagging. They therefore differ slightly in form from similar connections used in semi-continuous braced frames.

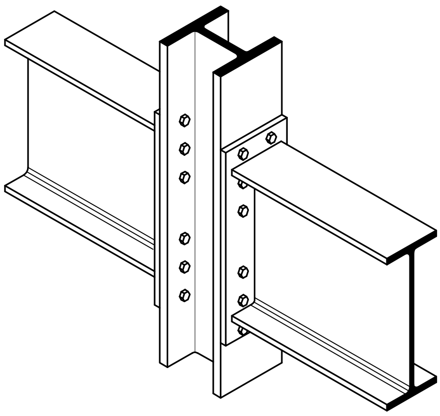

Figure 1.3 *Typical wind-moment connection*

Joints in steel construction: Moment connections [1] provides typical details, background design information and capacity tables for a range of standard connections that satisfy the required performance criteria. It also provides a full explanation of these criteria for wind-moment connections. Key points concerning the connections are as follows (with definitions of the various terms given schematically in Figure 1.4):

- The connections are *semi-rigid*; they possess a certain stiffness that is used primarily to limit the sway of the frame.
- The connections are *partial strength*; they possess some moment resistance, which is however less than the moment resistance of the adjoining beam.
- The connections are *ductile*; they can rotate as plastic hinges. Although the required rotation will vary for different frames, a value of 0.03 radians can be assumed to be sufficient for practical cases.

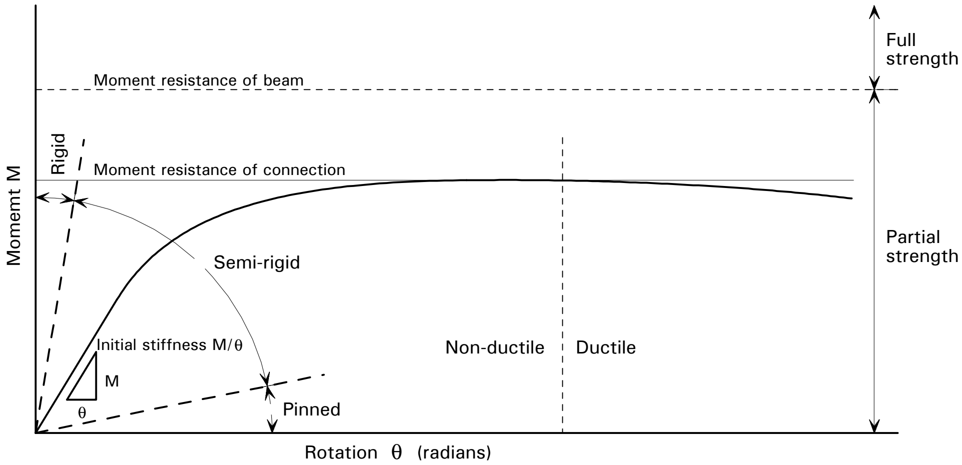

Figure 1.4 *Moment-rotation characteristics of a typical wind-moment connection*

In order to achieve these three characteristics, in standard *wind-moment* connections the end plates are thinner than those used in conventional fully rigid connections. Thin end plates are the most convenient and easily controlled means of achieving the required rotation because the end plate becomes the critical (i.e. weakest) connection component. However, it is important that the detailing of the standard connections given in Appendix D is adhered to strictly, as the choice of end plate thickness and bolt spacing in relation to the size and strength of the bolts, and of the welds, is crucial. If the end plate is too thin, both the stiffness and strength of the connection may be insufficient. If the end plate is too thick, the bolts or the welds may fail first, resulting in non-ductile behaviour.

1.3 Scope of this publication

1.3.1 Frame proportions

The wind-moment method has been validated by checking results for a broad range of low rise frames using non-linear finite element software [3,4,5]. In theory, the method could be used for a wider range of frames than those covered by the limitations given below. In order to simplify the procedures and remain within the bounds of the validation study, however, the method described in this publication should only be used for frames that comply with the following requirements:

- The geometry of the frame should be within the ranges shown in Table 1.1.
- The width of each bay should be constant over the height of the frame.
- The structure should be capable of being represented by a series of unbraced plane frames, each comprising a regular arrangement of orthogonal beams and columns.
- Beam layouts should comply with one of the options shown in Figures 1.5, 1.6 and 1.7.

When frames comply with the limits given above, use of the wind-moment method as described in this publication will lead to sections and connection details that are no *weaker* than those shown to be necessary during the background studies of similar frames[3,4,5]. This is an important point, and is the justification for the method described here.

Table 1.1 *Proportions of frames suitable for use of the wind-moment method*

Relative dimensions	Minimum	Maximum
Number of storeys	2	4
Number of bays	2	4*
Bay width (m)	4.5	12
Bottom storey height (m)	4.5	6
Storey height elsewhere (m)	3.5	5
Bay width: storey height (bottom storey)	0.75	2.5
Bay width: storey height (above bottom storey)	0.9	3
Greatest bay width: smallest bay width	1	2

* Frames may have more than 4 bays, but a core of 4 bays is the maximum that should be considered to resist the applied wind load. When adding non-active bays, the designer should remember that the notional horizontal loads applied to the core will increase.

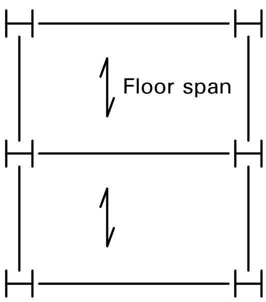

Figure 1.5 *Beam layout with floor spanning to* major axis *beams*

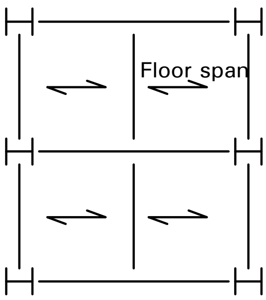

Figure 1.6 *Beam layout with floor spanning to secondary and* minor axis *beams*

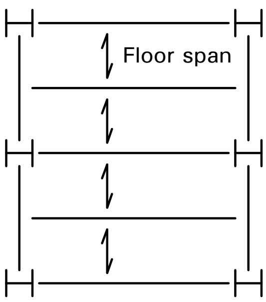

Figure 1.7 *Beam layout with floor spanning to intermediate and* major axis *beams*

1.3.2 Individual components

Individual frame components should comply with the following requirements in order for the frame design procedures given in this guide to remain valid:

- The steel grade may be either S275 or S355 (design grade 43 or 50) but the same design grade should be used for all the members in a given frame. For frames that are unbraced in the *minor axis* direction all sections should be S275 steel (grade 43).
- Horizontal members should be hot rolled Universal Beam or Universal Column sections, and should be able to meet the *plastic* or *compact* (Class 1 or 2) classification requirements of BS 5950-1 [6]. The inclusion of compact sections is a relaxation of the limits in SCI publication P082 *Wind-moment design for unbraced frames*.
- Members should not be composite [7].
- Vertical members should be hot rolled Universal Column sections and should be able to meet the *plastic* or *compact* (Class 1 or 2) classification requirements of BS 5950-1.
- Connections should be flush or extended end plates in accordance with the recommendations given for *wind-moment* connections in Appendix D.
- Columns should be rigidly connected to the foundations, by bases that are designed to resist moments.

1.3.3 Loading

Values of loading should not exceed the limits given in Table 1.2.

Wind loads may be derived using either CP3: Chapter V: Part 2 [8] or BS 6399-2 [9]. The resulting (unfactored) horizontal force at each floor level should not be taken as less than 10 kN. This level of loading corresponds to a wind speed of 37 m/s according to CP3 (for a gust) or 20 m/s according to BS 6399 (for an hourly mean).

Table 1.2 *Frame loading limits*

	Minimum	Maximum
Dead load on floors (kN/m^2)	3.5	5
Imposed load on floors (kN/m^2)	4	7.5
Dead load on roof (kN/m^2)	3.75	4
Imposed load on roof (kN/m^2)	1.5	1.5
Wind loads (kN)	10	40

The designer can determine rapidly whether the wind-moment method will be appropriate for a given frame by using the flowchart provided in Figure 1.8. If the frame is likely to be controlled by serviceability limit state (SLS) sway deflections, an alternative design method should be used.

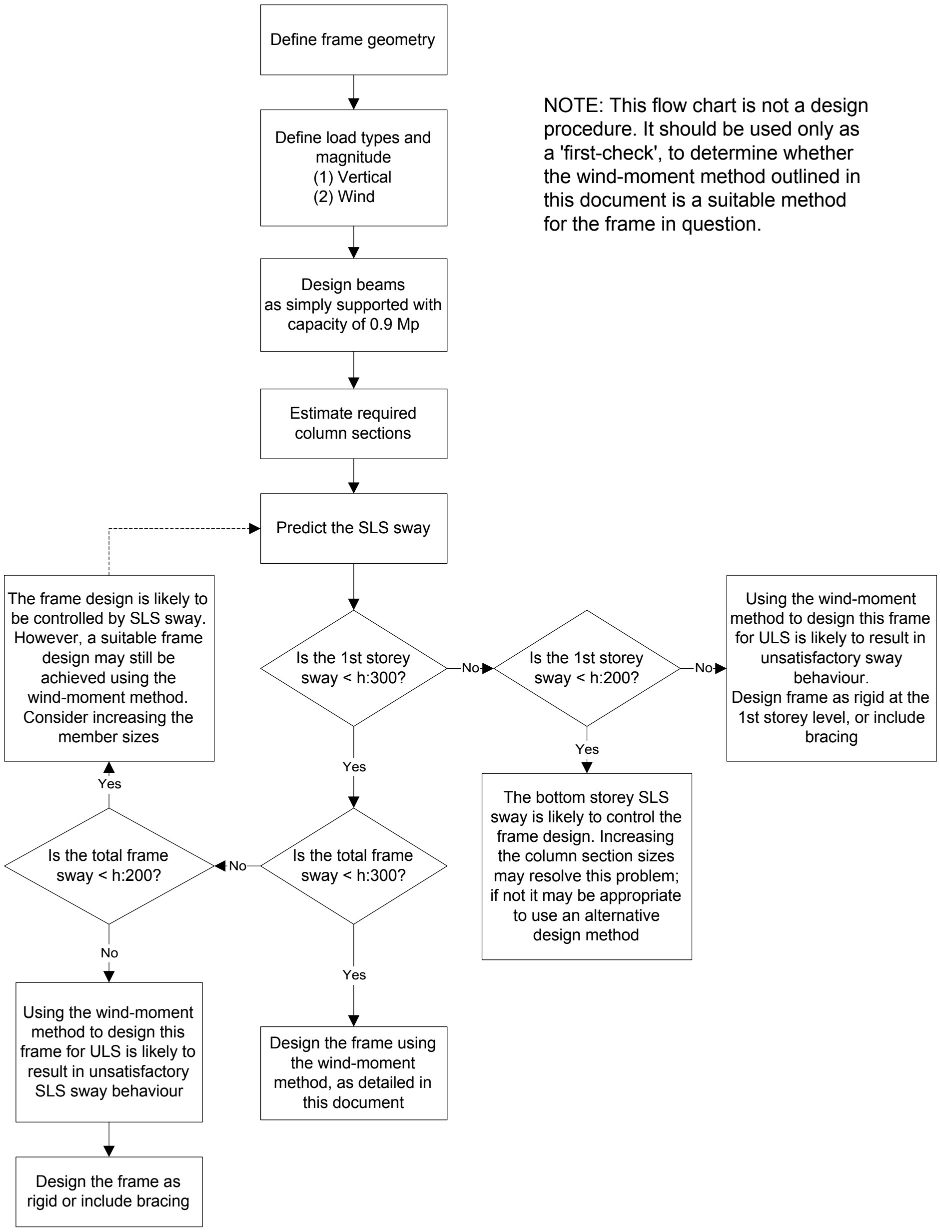

Figure 1.8 *Flow chart to determine the suitability of the wind-moment method for a given* major axis *frame*

2 DESIGN OF *MAJOR AXIS* FRAMES

2.1 Range of application

The wind-moment design procedures given in this Section apply to frames that are effectively braced at the roof and each floor level to prevent sway about the minor axes of the columns but are unbraced about the major axes of the columns (see Figure 2.1). These are referred to as *major axis* frames for simplicity. The prevention of sway about the minor axes can be achieved by cross bracing or by other systems such as attachment to a rigid core.

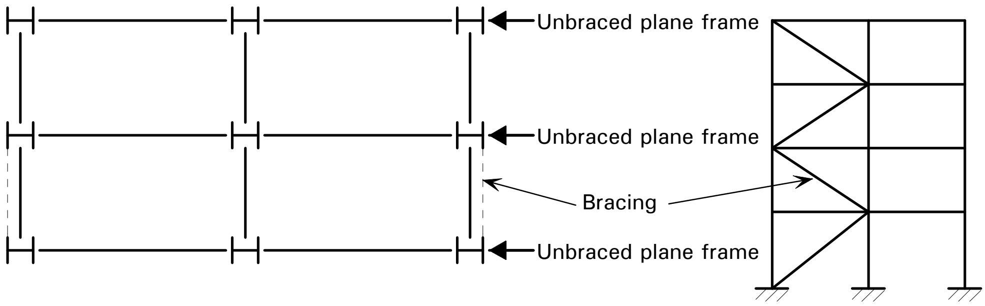

Figure 2.1 *Plane frames braced against out-of-plane sway*

Frame layouts, component details and loading should comply with the limits given in Section 1.3.

2.2 Global analysis at the ultimate limit state

2.2.1 Load combinations

The following load combinations, using load factors from BS 5950-1 [6], should be considered:

1.4 × Dead load + 1.6 × Imposed load + 1.0 × Notional horizontal forces
1.2 × Dead load + 1.2 × Imposed load + 1.2 × Wind load
1.4 × Dead load + 1.4 × Wind load

The notional horizontal forces should be taken as 0.5% of the factored dead plus imposed loads (BS 5950-1:1990: Clauses 5.6.3, 5.1.2.3) [6].

Pattern loading should be considered in addition to full gravity load on all beams.

2.2.2 Internal moments and forces due to vertical load

The internal moments and forces due to vertical load should be calculated in accordance with the requirements of BS 5950-1 for simple construction, with some slight modifications, as follows:

Beams

When calculating the moments in the beams due to vertical load, allowance should be made for the partial fixity of the beam to column connection by taking an end restraint moment equal to 10% of the free bending moment in the beam (i.e. 10%

of the sagging moment applied to the beam, assuming it to be simply supported). Consequently, the maximum sagging moment in the beam may be taken as 90% of the value calculated for a simply supported beam.

Columns

The moments in the columns due to vertical load alone are given by the algebraic sum of:

- The end restraint moments from the beams, which are taken as equal to 10% of the free bending moments in the beams (as described above).
- The moments due to eccentricity of the beam reactions, assuming that the reactions act 100 mm from the faces of the columns.

Account should be taken of the effects of pattern loading, if appropriate.

The net moment applied to a column at any one level should be divided between the column lengths above and below the level in proportion to the stiffness of those lengths (EI/L). When the ratio of stiffnesses does not exceed 1.5, the moment may be divided equally. These moments should be assumed to have no effect above and below the level at which they are applied.

Axial loads should be calculated considering both pattern loading and full loading, and combined with the appropriate moments.

Connections

The moments in the connections due to vertical loads should be calculated from the end restraint moments on the beams, as described above.

The shear forces in the connections due to vertical loads are given by the end reactions of the beams.

2.2.3 Internal moments and forces due to horizontal loads

The horizontal loading consists of the wind loading and the notional horizontal forces, as required by BS 5950-1. The notional forces, which are proportional to the floor loading, are used to model the effects of imperfections and lack of verticality of the members, and to ensure a minimum level of frame stability.

The frame should be analysed using the *portal method* (as described in Appendix A) or another established method to determine the applied forces and moments due to horizontal loads.

The portal method is based on the following assumptions:

- Horizontal loads are applied at floor levels.
- There is a point of contraflexure at the mid height of each column.
- There is a point of contraflexure at the mid length of each beam.
- Each bay acts as a simple portal, and the total horizontal load is divided between the bays in proportion to the span of each bay.

The bending moments resulting from these assumptions are shown in Figure 2.2. As the horizontal loads may reverse, the total moment at any point should be calculated by addition of the numerical values of the component moments.

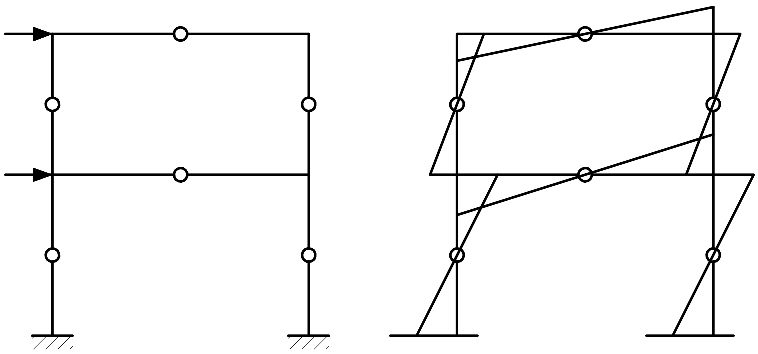

Figure 2.2 *Internal moments according to the portal method*

2.3 Design of beams at the ultimate limit state

Sections should be either Universal Beam or Universal Column sections that are classified as *plastic* or *compact* according to BS 5950-1.

The design moment capacity should be limited to 90% of the plastic moment resistance of the section in order to provide sufficient rotational restraint to the columns (BS 5950-1:1990: Clause 4.7.7). Because a nominal allowance of 10% is made for the partial fixity of the beam to column connection, the following relationship should therefore be satisfied:

$0.9\, M \# 0.9\, p_y\, S_x$

or alternatively $M \# p_y\, S_x$

where M is the applied moment due to vertical loading, assuming the beam is simply supported

p_y is the design strength of the steel

S_x is the plastic modulus of the beam about the major axis.

For parts of beams that are effectively unrestrained according to BS 5950-1, the following additional requirement should be satisfied;

$\bar{M} \# M_b$

where $\bar{M}$ is the equivalent uniform moment (see BS 5950-1:1990: Clause 4.3.7.2)

M_b is the lateral torsional buckling resistance moment (see BS 5950-1:1990: Clause 4.3.7.3).

In practical cases of beams directly supporting slabs, the beam is fully restrained by the slab, so there is no need to check lateral torsional buckling resistance.

2.4 Design of columns at the ultimate limit state

Sections should be Universal Column sections that are classified as *plastic* or *compact* according to BS 5950-1.

Effective lengths for compression resistance (P_c)

For in-plane buckling (i.e. buckling about the major axis of the section), the effective length L_E should be taken as $1.5\,L$.

For out-of-plane buckling (i.e. buckling about the minor axis), the effective length L_E should be taken as $1.0\,L$ when the columns are braced to prevent minor axis sway at each floor and roof level. With some bracing layouts, for example one in which the mid-height of the columns is also restrained, a shorter effective length could be envisaged.

Equivalent slenderness for lateral torsional buckling

The slenderness considered when calculating the lateral torsional buckling resistance M_b of the column should be taken as $0.5\,(L/r_y)$, where L is the length of the columns between restraints about both axes and r_y is the radius of gyration about the minor axis.

Interaction between moments and forces

The following relationship should be satisfied:

$$F_c\,/\,P_c + M_x\,/\,M_{bs} + M_y\,/\,p_y\,Z_y \; \# \; 1.0$$

where:
- F_c is the applied axial load due to vertical loading, or a combination of vertical loads and wind loads
- M_x is the applied moment about the major axis due to appropriate combinations of vertical loading, notional horizontal forces and wind loads
- M_y is the applied moment about the minor axis due to appropriate combinations of vertical loading
- p_y is the design strength of steel
- Z_y is the elastic modulus about the minor axis
- P_c is the compressive resistance
- M_{bs} is the lateral torsional buckling resistance moment for simple design

(P_c and M_{bs} should be calculated in accordance with BS 5950-1 using the effective lengths given above).

2.5 Design of connections at the ultimate limit state

The moments and forces applied to the connections due to vertical and horizontal loading are described in Section 2.2.

Appropriate connections should be chosen by comparing the moments and forces obtained from the analysis with the tabulated capacities given in Appendix D for the standard wind-moment details.

2.6 Column base design

Columns should be rigidly connected to the foundations by bases designed in accordance with the usual practice for nominally rigid details. Foundations should be designed to resist the combinations of axial load and bending moment that are given by the global frame analysis. Design guidance on column bases may be found in *Joints in steel construction: Moment connections* [1].

2.7 Serviceability limit state

BS 5950-1 lists the various requirements for design at the serviceability limit state. The only requirement that requires specific consideration in this guide is that of horizontal deflection (sway).

The deflection limits given in BS 5950-1 are not intended to be compared with the true deflections of the final structure but rather with those deflections calculated for the bare frame. They are based on practical experience and are values that, in general, will ensure that the resistance and in-service performance of the structure are not impaired. Examples of poor performance are visible deflections and cracking of brittle cladding materials and finishes. A sensible limit on horizontal deflection for low-rise frames is height/300. This limit is given in BS 5950-1.

2.7.1 Beam deflections

The vertical deflections of beams should generally be calculated using unfactored imposed loads assuming that the beams are simply supported. The limits on imposed load deflection should generally be in accordance with BS 5950-1: span/360 for beams carrying plaster or other brittle finishes.

If necessary, it may be possible to allow for the beneficial effects of the restraint offered by the connections when calculating beam deflections. Guidance may be found in *Design of semi-continuous braced frames* [10].

2.7.2 Horizontal deflections

The frame should be checked for sway using the unfactored wind loads and, where appropriate, any asymmetric loading that may cause sway.

Full analysis of frames taking into account connection flexibility shows that frames with *wind-moment* connections deflect significantly more under horizontal loading than those with fully rigid connections. This increased sway can be allowed for by the designer by means of a simple amplification factor applied to the sway deflections, as described below.

The simple graphical method outlined in Appendix B will generally be sufficiently accurate for calculating rigid frame deflections, although it should be noted that this method does not predict sway due to asymmetric vertical loads. The calculated *rigid frame* deflections should then be increased by 50% (i.e. multiplied by a factor of 1.5) as an approximate allowance for the flexibility of standard *wind-moment* connections (provided that the average bay width is at least 6 m). For an average bay width of 4.5 m, the deflections obtained from the rigid frame analysis should be multiplied by a factor of 2. For bay widths between 4.5 m and 6 m, linear interpolation between 2.0 and 1.5 should be used.

Where accurate calculation of deflections is critical, for example to ensure satisfactory installation or performance of the cladding, the above method may not provide a sufficiently accurate estimate of the frame sway, and a more detailed analysis taking explicit account of the flexibility of the connections may be required. Such methods are generally more appropriate to analysis by computer and are therefore not dealt with further in this publication.

If the deflections are unacceptable, either member sizes can be increased or connections can be replaced by rigid details and the frame redesigned as a fully rigid frame.

3 DESIGN OF *MINOR AXIS* FRAMES

The recommendations given in this Section apply to frames that are not braced in either principal direction. The rules concern primarily the *minor axis* frames (see Figure 3.1) although it should be noted that the *major axis* frame design will also be affected by the absence of bracing. A worked example illustrating the procedure to be adopted may be found in Appendix C.

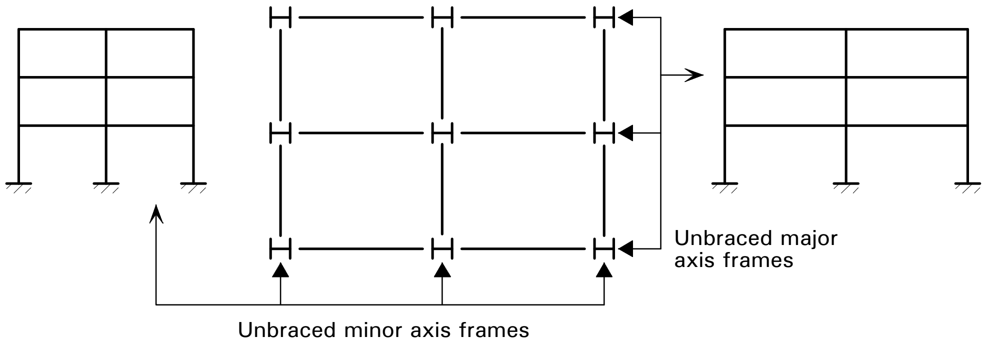

Figure 3.1 *Frame unbraced about both column axes*

The *wind-moment* design procedures outlined in Section 2 for *major axis* frames should be modified for this case, as follows:

- C The beams framing into the minor axis of the columns should be checked to ensure that they provide adequate stiffness for frame stability.
- C Columns must be checked to ensure that they provide adequate stiffness about their minor axis to ensure frame stability.
- C The connections to the minor axis should be in accordance with the provisions of Section 3.1.3.
- C Minor axis frames must be checked to ensure that, at the serviceability limit state, deflections satisfy the limits given in BS 5950-1.

Frames and components should also comply with the requirements of Section 1.3.

3.1 Design at the ultimate limit state

The frames should be designed to resist loading as discussed in Section 2.2, taking into account the additional moments about the minor axis of the columns due to the notional horizontal forces and wind loads. In addition, the stiffnesses of the beams used in the *minor axis* frames must satisfy certain requirements with respect to the column stiffnesses (see Section 3.1.2). As this check of relative stiffnesses will often govern the choice of section size, particularly if the applied gravity load on the beams is small, it is recommended that the following design procedure is adopted:

- C Design the columns using effective lengths as given in Section 3.1.1.
- C Size the beams for the *minor axis* frames so that the sum of the major axis stiffnesses of the beams meeting at a node is not less than the sum of the minor axis stiffnesses of the columns meeting at that node (see Section 3.1.2).

C Check the minor axis beams for appropriate combinations of gravity load, notional horizontal forces and wind load.

C Size the beams for the major axis frames using the procedures given in Section 2.

3.1.1 Design of columns

The forces and moments in the columns should be determined in the same way as for *major axis* frames (see Section 2.2), except that when considering pattern loading, an additional pattern that would induce the maximum moment about the minor axis should also be considered. Note that the wind loads and notional horizontal forces should only be considered to act in one direction at a time. It is generally more critical when these forces act on the *minor axis* frames, producing bending about the minor axis of the columns.

The design of the columns should be carried out using the procedures given in Section 2.4, except that the slenderness used to calculate the compressive resistance (P_c) about both the major and the minor axes of the columns should be based on an effective length of 1.5 L.

3.1.2 *Minor axis* beam design

Minor axis *beam stiffness*

In order to ensure that the minor axis beams have adequate stiffness to stabilise the frame, the relative beam to column stiffnesses should satisfy the following criteria (with reference to Figure 3.2):

At node 1:	$(I_{bx})_1 / L_1$	\$	$(I_{cy})_1 / h_1$					
At node 2:	$(I_{bx})_2 / L_1$	\$	$(I_{cy})_1 / h_1$	+	$(I_{cy})_2 / h_2$			
At node 3:	$(I_{bx})_3 / L_3$	\$	$(I_{cy})_2 / h_2$	+	$(I_{cy})_3 / h_3$			
At node 4:	$(I_{bx})_4 / L_4$	\$	$(I_{cy})_3 / h_3$	+	$(I_{cy})_4 / h_4$			
At node 5:	$(I_{bx})_1 / L_1$	+	$(I_{bx})_5 / L_2$	\$	$(I_{cy})_5 / h_1$			
At node 6:	$(I_{bx})_2 / L_1$	+	$(I_{bx})_6 / L_2$	\$	$(I_{cy})_5 / h_1$	+	$(I_{cy})_6 / h_2$	
At node 7:	$(I_{bx})_3 / L_1$	+	$(I_{bx})_7 / L_2$	\$	$(I_{cy})_6 / h_2$	+	$(I_{cy})_7 / h_3$	
At node 8:	$(I_{bx})_4 / L_1$	+	$(I_{bx})_8 / L_2$	\$	$(I_{cy})_7 / h_3$	+	$(I_{cy})_8 / h_4$	

and similarly across the rest of the frame. The positions of nodes 1 to 8 are shown in Figure 3.2. I_{bx} is the second moment of area of a beam about its major axis. I_{cy} is the second moment of area of a column about its minor axis. The subscripts 1 to 8 refer to various beam and column lengths as shown in Figure 3.2.

Although issues of relative stiffness are also relevant to *major axis* frames, an explicit check as described above is not necessary as the magnitude of vertical load applied to these beams ensures that they are adequately stiff for practical cases.

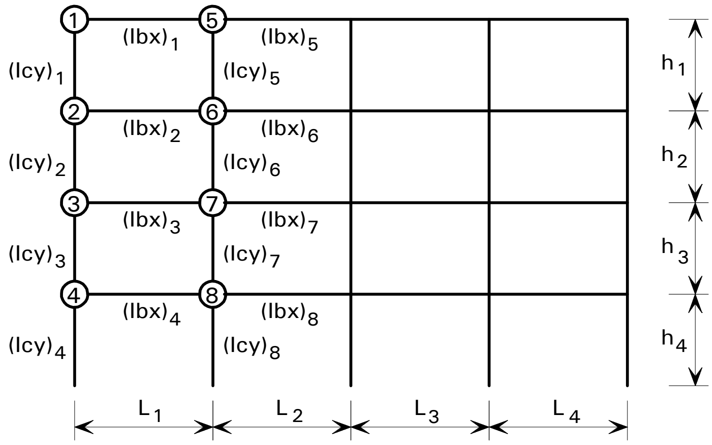

Figure 3.2 *Notation used to define minimum beam stiffness requirements*

Minor axis *beam resistance*

Beams should be designed for the moments and shears that arise due to appropriate combinations of vertical load, wind loads and notional horizontal forces. When the beam layout is as shown in Figures 1.6 and 1.7, gravity loading will generally govern the size of the *minor axis* beams. For a beam layout as shown in Figure 1.5, however, the gravity loading on the beams will be negligible, and horizontal loading will govern the required strength of the *minor axis* beams. In such cases, beam stiffness requirements are likely to dictate the final choice of section size.

For any beam layout, the plastic resistance and the lateral torsional buckling resistance of the beams framing into the minor axis should be calculated using the procedures given in Section 2.3.

3.1.3 Connection design

Minor axis connections must be detailed so that the following criteria are satisfied:

- The column side components should be relatively stiff so that they do not add significantly to the connection flexibility.
- Access can be gained to all welds, minor axis connection bolts and major axis connection bolts.
- Connection moment resistance is not governed by weld or bolt strength.

One potential way of detailing the minor axis connections is shown schematically in Figure 3.3. Standard *beam side* detailing is adopted in order to achieve the necessary flexibility and ductility, and doubler plates are used to provide the necessary web stiffness and strength. The plates are needed to enable moment transfer into the column without the column web simply distorting locally (although it may be difficult to transfer substantial moments with this type of detail). Beams may require notching depending on the relative dimensions of the members. Although it may be possible to achieve the necessary stiffness and strength with a doubler plate on one side only, attention should be paid to the thickness of the plate(s) to ensure there is no clash with the *major axis* bolts.

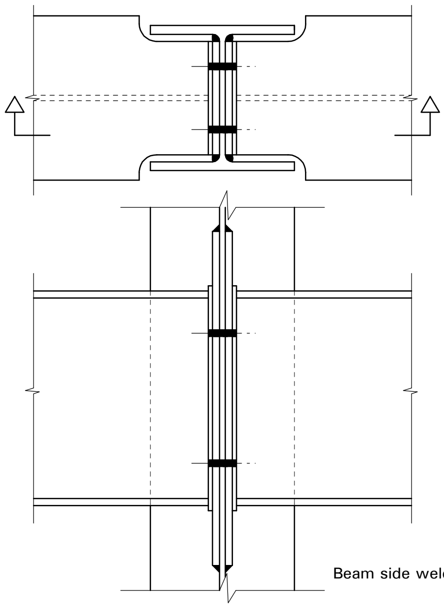

Figure 3.3 Minor axis *connection into a stiffened column web*

Specific weld requirements may differ from those shown in Figure 3.3 but it should be noted that the structural welds must be designed to avoid premature, non-ductile failure. According to EC3, this means designing the welds in an unbraced frame to be 70% over-strength. British practice suggests that full strength welds are sufficient [1].

Alternative connection details are currently being considered by The Steel Construction Institute and it is envisaged that further guidance will be published once a validatory test programme has been completed.

3.2 Design at the serviceability limit state

The vertical deflection of beams framing into both the major and minor axes of the columns should be checked (see Section 2.7.1). Sway deflections about both the major axis and the minor axis of the columns should be calculated using the procedures given in Section 2.7.2, and compared with a limit of 1/300 of the storey height in each storey.

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APPENDIX A PORTAL METHOD OF ANALYSIS

A.1 Introduction

The forces and moments in a multi-storey, multi-bay wind-moment frame can be determined by simple manual calculation using the so-called portal method. The wind and notional horizontal forces are shared between the bays according to the relative bay widths, and the forces in the beams and columns are calculated for this distribution of loading. A detailed explanation is given below for part of a multi-storey two-bay frame.

A.2 Distribution of horizontal load

Each bay is assumed to act as a single portal and the total horizontal load is divided between the bays in proportion to their spans. For a two bay frame, the loads in the two separate bays (as shown in Figure A.1b) are given by:

$$H_{1,1} = L_1 W_1 /(L_1 + L_2); \qquad H_{2,1} = L_2 W_1 /(L_1 + L_2)$$
$$H_{1,2} = L_1 W_2 /(L_1 + L_2); \qquad H_{2,2} = L_2 W_2 /(L_1 + L_2) \qquad \text{(A.1)}$$

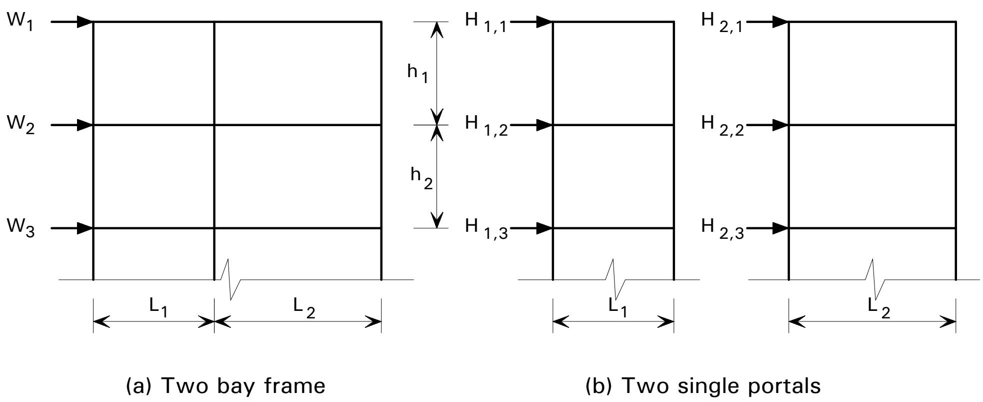


Figure A.1 *Distribution of horizontal load*

A.3 Calculation of internal forces in columns

The forces acting on a part of one bay and the pin locations assumed in wind-moment design are shown in Figure A.2a.

The forces acting on the portion of the bay above the points of contraflexure at A and D are shown in Figure A.2b. The horizontal force H_1 is assumed to be divided equally between the two columns. Thus

$$S_1 = H_1 / 2 \qquad \text{(A.2)}$$

The vertical forces F_1 can be found by taking moments about the point of contraflexure at either A or D:

$$F_1 L = H_1 h_1 / 2$$

which gives:

$$F_1 = H_1 h_1 / (2L) \quad \text{(A.3)}$$

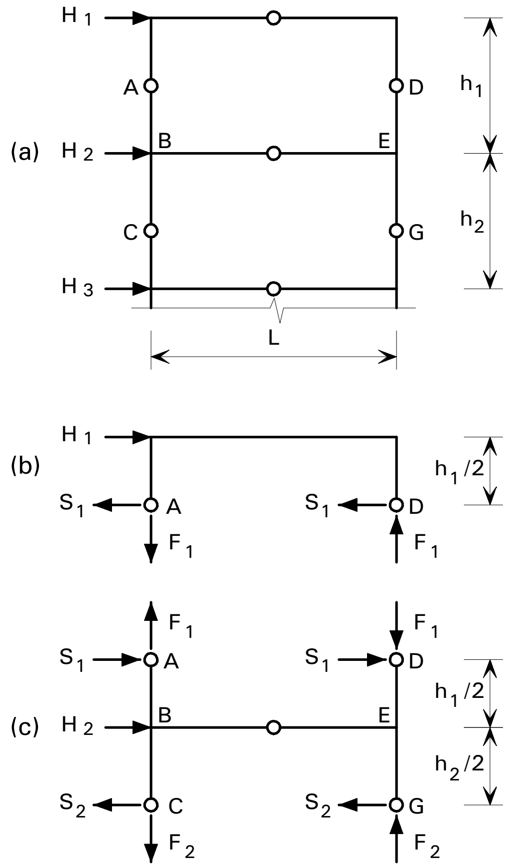


Figure A.2 *Internal forces in columns*

The forces acting on the portion ABCDEG of the bay are shown in Figure A.2c. It follows from the assumption above that:

$$S_2 = (H_1 + H_2)/2 \quad \text{(A.4)}$$

Taking moments about the point of contraflexure at either C or G:

$$F_2 L = H_2 h_2 /2 + 2 S_1 (h_1 + h_2)/2 + F_1 L$$

Substituting for S_1 and F_1 and re-arranging:

$$F_2 = H_1 h_1 /L + (H_1 + H_2) h_2 / (2L) \quad \text{(A.5)}$$

A.4 Calculation of internal moments

It is clear from Figure A.2b that the internal moment at the head of each column is given by:

$$M_1 = S_1 h_1 /2$$

Substituting for S_1:

$$M_1 = H_1 h_1 / 4 \tag{A.6}$$

For equilibrium, the moment at each end of the roof beam is also equal to M_1. The bending moment diagram is shown in Figure A.3a.

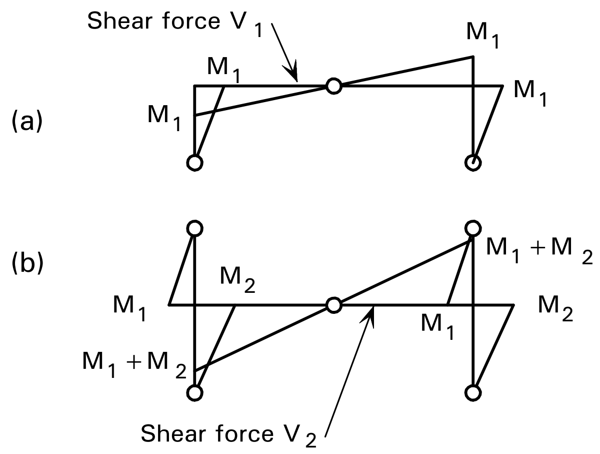


Figure A.3 *Internal moments*

Referring to Figure A.2c, the internal moment in each upper column at B and E is also M_1. The corresponding moment in the lower columns is given by:

$$M_2 = S_2 h_2 / 2$$

Substituting for S_2:

$$M_2 = (H_1 + H_2) h_2 / 4 \tag{A.7}$$

For equilibrium at B and E, the internal moment at each end of the beam BE equals ($M_1 + M_2$), as shown in Figure A.3b.

A.5 Calculation of shear forces in beams

As a point of contraflexure is assumed at the mid-length of each beam (Figure A.3), the shear force in the roof beam is given by:

$$V_1 = M_1 / (L/2)$$

Substituting for M_1:

$$V_1 = H_1 h_1 / (2L) \tag{A.8}$$

Similarly, the shear force V_2 in beam BE is given by:

$$V_2 = (M_1 + M_2) / (L/2)$$

Substituting for M_1 and M_2:

$$V_2 = H_1 (h_1 + h_2) / (2L) + H_2 h_2 / (2L) \tag{A.9}$$

A.6 Forces and moments in an internal column

These are obtained by summing the values calculated for adjacent bays on either side of the column.

It is found that the vertical forces in an internal column due to horizontal loading are zero.

APPENDIX B WORKED EXAMPLE: *MAJOR AXIS* FRAME

B.1 Introduction

The design example given in this Appendix is for a frame that is braced out of plane in order to prevent sway about the minor axis of the columns.

Calculations are given to demonstrate the following aspects of the design rules:

- compliance with the scope of the method
- framing and loads
- wind analysis
- notional horizontal forces and analysis
- beam design
- column loads
- internal column design
- external column design
- connections design
- serviceability limit state.

B.1.1 Compliance

The frame in this example forms part of a steel structure that conforms to the frame layout specified in Section 1.3 above. In particular:

- The frame is effectively braced against sway about the minor axis of the columns at roof level and each floor level.
- The floor layout comprises only primary beams, with flooring and roofing spanning as shown in Figure 1.5.

B.1.2 Frame dimensions

The frame dimensions conform to the range of application specified in Section 1.3:

bay width : storey height (bottom storey)	=	$\frac{6}{5}$	=	1.2
bay width : storey height (above bottom storey)	=	$\frac{6}{4}$	=	1.5
greatest bay width : smallest bay width	=	$\frac{6}{6}$	=	1.0
storey height (bottom storey)	=	5 m	<	6 m
storey height (other storeys)	=	4 m	<	5 m

B.1.3 Loading

The following unfactored loading conforms to the range of application given in Section 1.3.3:

dead load on roof = 4.00 kN/m^2 (between 3.75 and 4.0 kN/m^2, OK)

imposed load on roof = 1.50 kN/m^2 (equal to 1.5 kN/m^2, OK)

dead load on floor = 4.50 kN/m^2 (between 3.5 and 5.0 kN/m^2, OK)

imposed load on floor = 5.00 kN/m^2 (between 4.0 and 7.5 kN/m^2, OK)

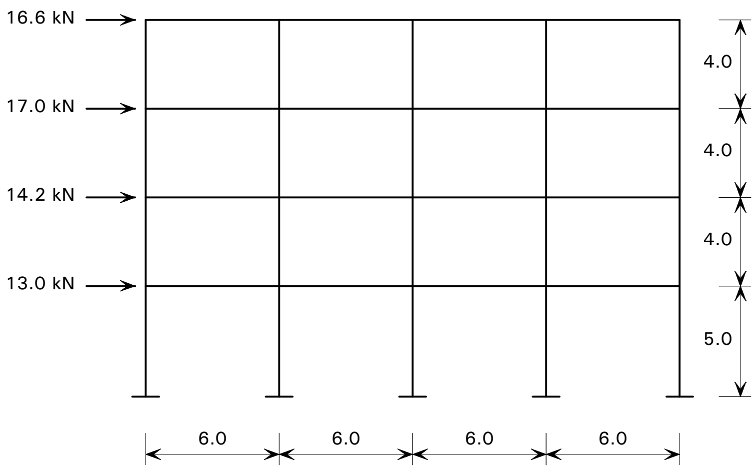
wind forces (see Figure B.1) equate to a basic wind speed that is not less than 37 m/s (to CP3: Chapter V: Part 2).

B.1.4 Design

The members have been designed using the rules given in BS 5950-1:1990 [3], with additional requirements as specified in this publication.

Member capacities were obtained directly from the capacity tables given in the SCI publication *Steelwork design guide to BS 5950: Part 1: 1990 - Volume 1: Section properties and member capacities* [11].

B.2 Framing and loads



Frames located at 6.0 m centres longitudinally

Figure B.1 Frame arrangement and applied wind loads

Roof

Dead load	*4.0 kN/m²*	*24.0 kN/m*
Imposed load	*1.5 kN/m²*	*9.0 kN/m*

Floors

Dead load	*4.5 kN/m²*	*27.0 kN/m*
Imposed load	*5.0 kN/m²*	*30.0 kN/m*

B.3 Wind analysis

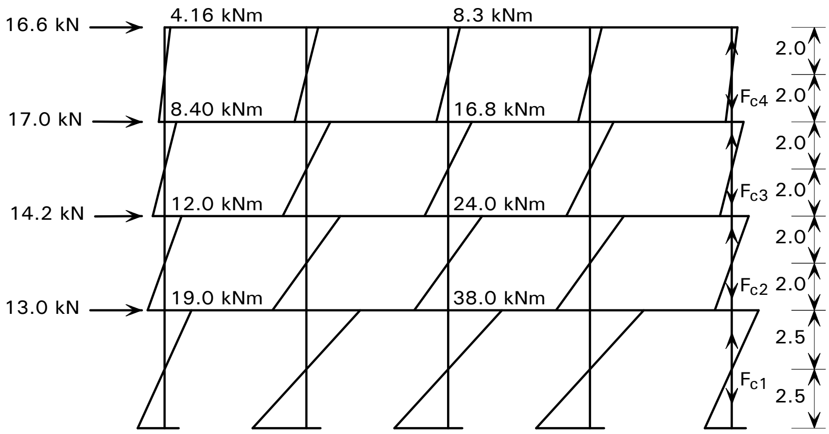


Figure B.2 Frame analysis under wind loads (columns)

Table B.1 Shear forces and bending moments in the column due to wind load

Storey	Total wind shear (kN)	Shear force in column (kN)		Bending moment in columns (kNm)	
		External	Internal	External	Internal
4	16.6	2.08	4.15	2.08 × 2.0 = 4.16	4.15 × 2.0 = 8.30
3	33.6	4.2	8.4	4.20 × 2.0 = 8.40	8.40 × 2.0 = 16.8
2	47.8	5.98	12	5.98 × 2.0 = 12.0	12.0 × 2.0 = 24.0
1	60.8	7.6	15.2	7.60 × 2.5 = 19.0	15.2 × 2.5 = 38.0

Table B.2 Moments and axial loads due to wind loads

Storey	Moments about point of contraflexure at mid-height	F_c (kN)
4	$24\ F_{c4} = 16.6 \times 2.0$	1.38
3	$24\ F_{c3} = 16.6 \times 6.0 + 17.0 \times 2.0$	5.6
2	$24\ F_{c2} = 16.6 \times 10.0 + 17.0 \times 6.0 + 14.2 \times 2.0$	12.4
1	$24\ F_{c1} = 16.6 \times 14.5 + 17.0 \times 10.5 + 14.2 \times 6.5 + 13.0 \times 2.5$	22.7

N.B. Values are UNFACTORED

Axial forces in the beams due to wind loads are small and may be neglected.

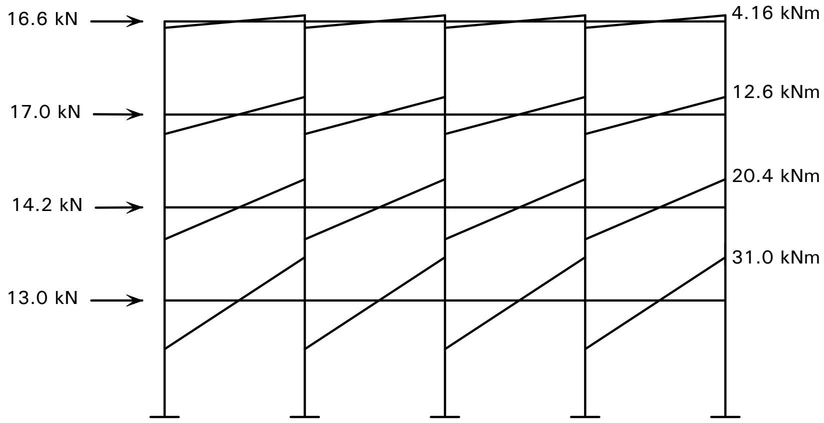


Figure B.3 Frame analysis under wind loads (beams)

Table B.3 Bending moments in the beams due to wind loads

Floor level	Bending moment in external columns (kNm)		Bending moment in beams (kNm)
	Upper column	Lower column	
Roof	-	4.16	0.0 + 4.16 = 4.16
3	4.16	8.4	4.16 + 8.40 = 12.6
2	8.4	12	8.40 + 12.0 = 20.4
1	12	19	12.0 + 19.0 = 31.0

N.B. Values are UNFACTORED

B.4 Notional horizontal forces and analysis

Notional horizontal force = 0.005 (1.4 Dead + 1.6 Imposed)

Roof H $= 0.005\ (1.4 \times 24 + 1.6 \times 9) \times 24 = 5.76$ kN

Floor H $= 0.005\ (1.4 \times 27 + 1.6 \times 30) \times 24 = 10.3$ kN

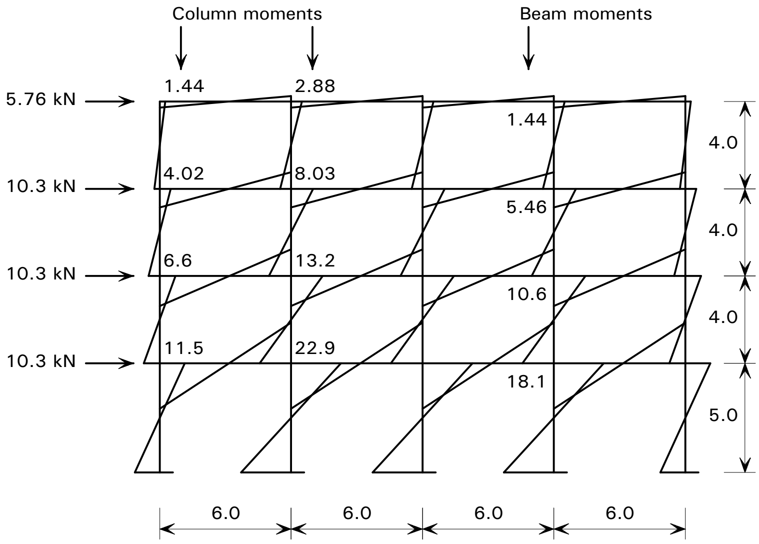


Figure B.4 Frame analysis due to notional horizontal forces

Table B.4 Shear forces and bending moments in the column due to notional horizontal forces

Storey	Total shear (kN)	Shear in column (kN)		Bending moment in column (kNm)	
		External	Internal	External	Internal
4	5.76	0.72	1.44	0.72 × 2 = 1.44	1.44 × 2 = 2.88
3	16.1	2.01	4.01	2.01 × 2 = 4.02	4.01 × 2 = 8.03
2	26.4	3.3	6.6	3.30 × 2 = 6.60	6.60 × 2 = 13.2
1	36.7	4.58	9.16	4.58 × 2.5 = 11.5	9.16 × 2.5 =22.9

Table B.5 Bending moments in the beams due to notional horizontal forces

Floor level	Bending moment in external column (kNm)		Bending moment in beam (kNm)
	Upper column	Lower column	
Roof	-	1.44	0.0 + 1.44 = 1.44
3	1.44	4.02	1.44 + 4.02 = 5.46
2	4.02	6.6	4.02 + 6.60 = 10.6
1	6.6	11.5	6.60 + 11.5 = 18.1

N.B. Values are FACTORED as the notional horizontal force is a percentage of the factored floor load.

The axial forces generated in the beams and columns by the notional horizontal forces are small and may be neglected.

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B.5 Beam design

B.5.1 Roof beam

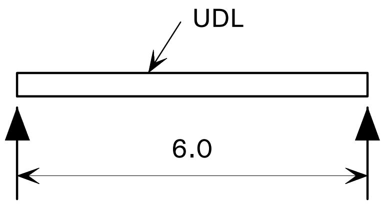

Figure B.7 Roof beam

Ultimate limit state

Dead load plus imposed loading

Design load for ULS: $W = (1.4 \times 24 + 1.6 \times 9) \times 6 = 288$ kN — *Sheet 1*

Taking advantage of the 10% restraint moment at the end of the beams

The maximum moment at the centre of the span — *Section 2.3*

$$M = \frac{0.9\ WL}{8}\ ,\ \frac{0.9 \times 288 \times 6}{8} = 194 \text{ kNm}$$

The maximum shear force at the end of the span

$$F_y = \frac{W}{2}\ ,\ \frac{288}{2} = 144 \text{ kN}$$

Try 305 × 165 × 54 UB S275 steel

Section is Class 1 plastic — *OK*

The beam is fully restrained.

Moment capacity (M_{cx})

In order to provide directional restraint to the columns, the moment capacity is limited to 0.9 M_{cx}. — *Section 2.3*

$0.9\ M_{cx} = 0.9 \times 233 = 210 \text{ kNm} > 194 \text{ kNm}$ — *OK* — *Ref 11*

Shear capacity (P_v)

P_v = 405 kN > 144 kN OK — ***Ref 11***

<u>Dead load plus wind loading</u>

Design moment at end of beam due to wind = 1.4 × 4.2 = 5.9 kNm

By inspection, this load combination not critical

<u>Dead load plus imposed load plus wind loading</u>

Design moment at end of beam due to wind = 1.2 × 4.2 = 5.0 kNm

By inspection, this load combination not critical

<u>Serviceability limit state</u>

Design imposed load for SLS: W = 9.0 × 6 = 54 kN — ***Sheet 1***

Imposed load deflection of beam (assuming simply supported)

$$*_1 = \frac{5 \times 54 \times 6000^3}{384 \times 205 \times 11700 \times 10^4} \; ' \; 6.3\,mm \; ' \; \frac{Span}{950}$$ ***OK***

<u>Use 305 × 165 × 54 UB S275 steel</u>

B.5.2 Floor beam

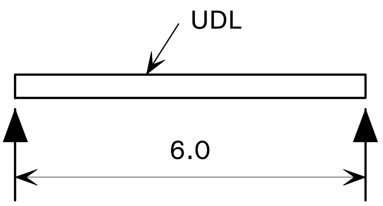

Figure B.8 Floor beam

<u>Ultimate limit state</u>

<u>Dead load plus imposed loading</u>

Design load for ULS: W = (1.4 × 27 + 1.6 × 30) × 6 = 515 kN — ***Sheet 1***

Taking advantage of the 10% restraint moment at the end of the beam

The maximum moment at the centre of the span

$$M = \frac{0.9\ WL}{8} \text{ , } \frac{0.9 \times 515 \times 6}{8} = 347 \text{ kNm}$$

The maximum shear at the end of the span

$$F_v = \frac{W}{2} \text{ , } \frac{515}{2} = 257 \text{ kN}$$

<u>Try 406 × 178 × 74 UB S275 steel</u>

The section is Class 1 plastic.

The beam is fully restrained.

Moment capacity (M_{cx})

In order to provide directional restraint to the columns, the moment capacity is limited to 0.9 M_{cx}.

$0.9\ M_{cx} = 0.9 \times 413 = 372 \text{ kNm} > 347 \text{ kNm}$ **OK** — *Ref 11*

Shear capacity (P_v)

$P_v = 647 \text{ kN} > 257 \text{ kN}$ **OK** — *Ref 11*

<u>Dead load plus wind loading</u>

Design moment at end of beam due to wind = 1.4 × 31.0 = 43.4 kNm — *Table B.3*

By inspection, this load combination not critical

<u>Dead load plus imposed load plus wind loading</u>

Design moment at end of beam due to wind = 1.2 × 31.0 = 37.2 kNm — *Table B.3*

By inspection, this load combination not critical.

<u>Serviceability limit state</u>

Design imposed load at SLS: $W = 30 \times 6 = 180 \text{ kN}$ — *Sheet 1*

Imposed load deflection of beam (assuming simply supported) =

$$\frac{5 \times 180 \times 6000^3}{384 \times 205 \times 27300 \times 10^4} = 9.05\,mm = \frac{Span}{660} < \frac{Span}{360} \qquad OK$$

Use 406 × 178 × 74 UB S275 steel

B.6 Column loads

Data for calculation of column moments are given in Table B.6.

Table B.6 Data for calculation of column moments

Storey	Beam reactions		10% restraint moment		Moments due to horizontal loads			
					Notional loads		Wind	
	Dead (kN)	Imposed (kN)	Dead (kNm)	Imposed (kNm)	External (kNm)	Internal (kNm)	External (kNm)	Internal (kNm)
3	81	90	12.2	13.5	4.02	8.03	8.4	16.8
1	81	90	12.2	13.5	11.5	22.9	19	38

Sheet 1

Table B.4

Table B.1

N.B. All values are UNFACTORED, except for moments due to notional horizontal loads

The values for the 10% restraint moment are calculated from the unfactored floor loads (i.e. 10% of $wL^2/8$)

Dead = $0.1 \times 27 \times 6^2/8 = 12.2$

Imposed = $0.1 \times 30 \times 6^2/8 = 13.5$

B.7 Internal column design

The columns will be spliced above the second storey floor beams, where change in section size may take place. Therefore, design calculations will be required for storeys 3 and 1.

Table B.7 Loading on internal columns

Storey	*Loading (kN)*	*Sw of column (kN)*	*Total load*		*Reduction in imposed load (kN)*	*Reduced imposed load (kN)*
			Dead (kN)	*Imposed (kN)*		
4	D 72 / I 27 — D 72 / I 27	3	147	54	0	54
3	D 81 / I 90 — D 81 / I 90	3	312	234	10% 23	211
2	D 81 / I 90 — D 81 / I 90	5	479	414	20% 83	331
1	D 81 / I 90 — D 81 / I 90	6	647	594	30% 178	416

N.B. Values are UNFACTORED

The reduction in imposed load for the number of storeys carried is given by BS 6399-1: Table 2.

B.7.1 Storey 3

<u>Dead load plus imposed load plus notional forces</u>

Design load at ULS: $F_c = 1.4 \times 312 + 1.6 \times 211 = 774$ kN — *Table B.7*

Design moment at ULS: $M_x = 8.03$ kNm — *Table B.6*
(due to notional loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis. By inspection, pattern imposed load (i.e. omitting imposed load on one beam at third floor level) will not be critical.

$L = 4.0$ m

$L_{Ey} = 1.0L = 4.0$ m; $L_{Ex} = 1.5L = 6.0$ m — *Section 2.4*

<u>Try 203 × 203 × 52 UC S275 steel</u>

Section is Class 1 plastic

At L_{EY} = 4 m $\quad P_{cy}$ = 1100 kN — *Ref 11*

At L_{EX} = 6 m $\quad P_{cx}$ = 1370 kN — *Ref 11*

At L = 4 m $\quad M_{bs}$ = 150 kNm — *Ref 11*

$$\frac{F_c}{P_c} \% \frac{M_x}{M_{bs}} \text{ ' } \frac{774}{1100} \% \frac{8.03}{150} \text{ ' } 0.76 < 1.00$$

OK — *Section 2.4*

By inspection, pattern imposed load will not be critical.

<u>Dead load plus imposed load plus wind loading</u>

Design load at ULS: $F_c = 1.2 \times 312 + 1.2 \times 211 = 628$ *kN* — *Table B.7*

Design moment at ULS: $M_x = 1.2 \times 16.8 = 20.2$ *kNm* — *Table B.6*

(due to notional loads)

Moments due to eccentric reactions and the 10% restraint moment balance and produce no net moment about the major axis.

$$\frac{F_c}{P_c} \% \frac{M_x}{M_{bs}} \text{ ' } \frac{628}{1100} \% \frac{20.2}{150} \text{ ' } 0.71 < 1.00$$

OK — *Section 2.4*

<u>Dead load plus wind loading</u>

Design load at ULS: $F_c = 1.4 \times 312 = 437$ *kN* — *Table B.7*

Design moment at ULS: $M_x = 1.4 \times 16.8 = 23.5$ *kNm* — *Table B.1*

$$\frac{F_c}{P_c} \% \frac{M_x}{M_{bs}} \text{ ' } \frac{437}{1100} \% \frac{23.5}{150} \text{ ' } 0.55 < 1.00$$

OK — *Section 2.4*

<u>Use 203 × 203 × 52 UC S275 steel</u>

B.7.2 Storey 1

<u>Dead load plus imposed loading plus notional forces</u>

Design load at ULS: $F_c = 1.4 \times 647 + 1.6 \times 416 = 1571$ *kN* — *Table B.7*

Design moment at ULS: $M_x =$ *22.9 kNm* — *Table B.6*

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$L = 5.0\ m$

$L_{Ey} = 1.0\ L = 5.0\ m;\quad L_{Ex} = 1.5L = 7.5\ m$ — *Section 2.4*

__Try 254 × 254 × 89 UC S275 steel__

Section is Class 1 plastic

$At\ L_{Ey} = 5\ m \qquad P_{cy} = 1860\ kN$ — *Ref 11*

$P_{cx} > P_{cy}$

$At\ L = 5\ m \qquad M_{bs} = 316\ kNm$ — *Ref 11*

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{1571}{1860} + \frac{22.9}{317} = 0.92 < 1.00$$

OK — *Section 2.4*

By inspection, pattern loading will not be critical.

__Dead load plus imposed load plus wind loading__

Design load at ULS: $F_c = 1.2 \times 647 + 1.2 \times 416 = 1276\ kN$ — *Table B.7*

Design moment at ULS: $M_x = 1.2 \times 38.0 = 45.6\ kNm$ — *Table B.1*

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{1276}{1860} + \frac{45.6}{317} = 0.83 < 1.00$$

OK

__Dead load plus wind loading__

Design load at ULS: $F_c = 1.4 \times 647 = 906\ kN$ — *Table B.7*

Design moment at ULS: $M_x = 1.4 \times 38.0 = 53.2\ kNm$ — *Table B.1*

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{906}{1860} + \frac{53.2}{317} = 0.65 < 1.00$$

OK — *Section 2.4*

__Use 254 × 254 × 89 UC S275 steel__

B.8 External column design

Table B.8 Loading on external columns

Storey	Loading (kN)	Sw of column (kN)	Total load		Reduction in imposed load (kN)	Reduced imposed load (kN)
			Dead (kN)	Imposed (kN)		
4	D 72, I 27	3	75	27	0	27
3	D 81, I 90	3	159	117	10% 12	105
2	D 81, I 90	5	245	207	20% 41	166
1	D 81, I 90	6	332	297	30% 89	208

N.B. Values are UNFACTORED

The reduction in imposed load for number of storeys carried is given by BS 6399-1: Table 2.

10% restraint moments

The moments due to partial fixity of the beam ends are taken from Table B.6.

Moment due to dead load = 12.2 kNm

Moment due to imposed load = 13.5 kNm

B.8.1 Storey 3

<u>*Dead load plus imposed load plus notional forces*</u>

Design load at ULS: $F_c = 1.4 \times 159 + 1.6 \times 105 = 391$ *kN* — *Table B.8*

Design moment at ULS: Mx

Assume section 200 deep

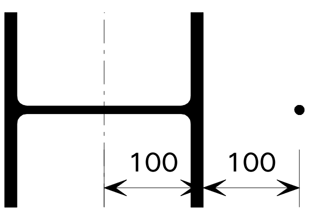

Figure B.9 External column (storey 3)

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Eccentricity moment (1.4 × 81 + 1.6 × 90) (0.1 + 0.1) =	*51.5 kNm*	*Table B.8*
10% restraint moment (1.4 × 12.2 + 1.6 × 13.5) =	*<u>38.7</u>*	
	90.2 kNm	
Divide moment equally between upper and lower column lengths	*45.1 kNm*	*Section 2.4*
Notional horizontal loads	*<u>4.0</u>*	*Table B.6*
Total design moment M_x =	*49.1 kNm*	
L = 4.0 m		
L_{Ey} = 1.0L = 4.0 m; L_{Ex} = 1.5L =	*6.0 m*	*Section 2.4*
<u>Try 203 × 203 × 52 UC S275 steel</u>		
Section Class 1 plastic		*Ref 11*
At L_{EY} = 4 m P_{cy} = 1100 kN		*Ref 11*
At L_{EX} = 6 m P_{cx} = 1370 kN		*Ref 11*
At L = 4 m M_{bs} = 150 kNm		*Ref 11*
$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} = \frac{391}{1100} + \frac{49.1}{150} = 0.68 < 1.00$	*OK*	*Section 2.4*
<u>Dead load plus imposed load plus wind loading</u>		
Design load at ULS: F_c = 1.2 (159 + 105 + 5.6) =	*324 kN*	*Tables B.8 and B.2*
Design moment at ULS: M_x		
Eccentricity moment (1.2 × 81 + 1.2 × 90) (0.1 + 0.1) =	*41.0 kNm*	*Table B.7*
10% restraint moment (1.2 × 12.2 + 1.2 × 13.5) =	*<u>30.8</u>*	
	71.8 kNm	
Divide moment equally between upper and lower column lengths	*35.9 kNm*	
Moment due to wind (1.2 × 8.4)	*<u>10.1</u>*	*Table B.1*

Total design moment M_x = *46.0 kNm*

$\frac{F_c}{P_c} \% \frac{M_x}{M_{bs}} \text{'} \frac{324}{1100} \% \frac{46.0}{150} \text{'} 0.6 < 1.00$ *OK* — *Section 2.4*

<u>Dead plus wind loading</u>

By inspection, not critical

<u>Use 203 × 203 × 52 UC S275 steel</u>

B.8.2 Storey 1

<u>Dead load plus imposed load plus notional forces</u>

Design load at ULS: $F_c = 1.4 \times 332 + 1.6 \times 208$ = *798 kN* — *Table B8*

Design moment at ULS: M_x

Assume section 200 deep

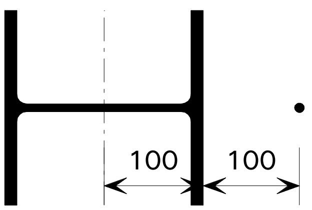

Figure B.10 External column (storey 1)

Eccentricity moment $M_x = (1.4\times81 + 1.6\times90)\ (0.1 + 0.1)$ = *51.5 kNm* — *Table B.8*

10% restraint moment $M_x = (1.4 \times 12.2 + 1.6 \times 13.5)$ = <u>*38.7*</u>

90.2 kNm

Divide equally between upper and lower column lengths = *45.1 kNm*

Moment due to notional horizontal loads M_x = <u>*11.5*</u> — *Table B.6*

Total design moment M_x = *56.6 kNm*

$L = 5.0\ m$

$L_{Ey} = 1.0L = 5.0\ m$; $L_{Ex} = 1.5L = 7.5\ m$ — *Section 2.4*

__Try 203 × 203 × 71 UC S275 steel__

At L_{EY} = 5 m P_{cy} = 1190 kN — *Ref 11*

$P_{cx} > P_{cy}$

At L = 5 m M_{bs} = 190 kNm — *Ref 11*

$$\frac{F_c}{P_c} \,\%\, \frac{M_x}{M_{bs}} \;'\; \frac{798}{1190} \,\%\, \frac{56.6}{190} \;'\; 0.97 < 1.00$$

OK — *Section 2.4*

__Dead load plus imposed load plus wind loading__

Design load at ULS: F_c = 1.2 (332 + 208 + 22.7) = 675 kN — *Tables B.8 and B.2*

Design moment at ULS: M_x

Eccentricity moment M_x = (1.2×81+1.2×90) (0.1+0.1) = 41.0 kN

10% restraint moment M_x = (1.2 × 12.2 + 1.2 × 13.5) = <u>30.8</u>

71.8 kNm

Divide equally between upper and lower column lengths 35.9 kNm

Wind load W_w = 1.2 × 19.0 = <u>22.8</u> — *Table B.1*

Total design moment M_x 58.7 kNm

$$\frac{F_c}{P_c} \,\%\, \frac{M_x}{M_{bs}} \;'\; \frac{675}{1190} \,\%\, \frac{58.7}{190} \;'\; 0.88 < 1.00$$

OK

__Dead load plus wind loading__

By inspection, not critical

__Use 203 × 203 × 71 UC S275 steel__

B.9 Connection design

Calculations are given for a connection to a perimeter column at first floor level only, as an example of how the design should be carried out.

Dead load plus imposed load plus notional forces

Design moment at ULS: $= (1.4 \times 12.2 + 1.6 \times 13.5) + 18.1$ — *Sheet 12, Table B.5*

(Moment from 10% beam restraint and notional horizontal loads) $= 56.8$ kNm

Design shear at ULS: $F_v = (1.4 \times 81 + 1.6 \times 90) + \dfrac{18.1}{3}$

(Shear from beam reactions and notional horizontal force) $= 263$ kN

Dead load plus imposed load plus wind loading

Design moment at ULS: $M = (1.2\times12.2 + 1.2\times13.5) + 1.2\times31$ — *Sheet 12, Table B.3*

(Moment from 10% beam restraint and wind loading) $= 68.0$ kNm

Design shear at ULS: $F_v = (1.2 \times 81 + 1.2 \times 90) + \dfrac{1.2 \times 31}{3}$

(Shear from beam reactions and wind loading) $= 218$ kN

Dead load plus wind loading

Design moment at ULS: $M = 1.4 \times 12.2 + 1.4 \times 31.0$

(Moment from 10% beam restraint and wind loading) $= 60.5$ kNm

Design shear at ULS: $F_v = 1.4 \times 81 + \dfrac{1.4 \times 31}{3}$

(Shear from 10% beam restraint and wind loading) $= 128$ kN

A suitable connection for a 406 × 178 × 74 UB to a 203 × 203 × 71 UC selected from Appendix D would be a 12 mm flush end plate with a single row of tension bolts, see page 67.

For this connection M_c = 70 kNm and the shear capacity with an extra bolt row = 442 kN, compared with the design values of 60.5 kNm and 128 kN respectively.

Column panel shear at ULS:

Having chosen a possible connection based on consideration of applied moment and shear, the column panel shear resistance must now be checked.

The critical moment is 68 kNm, due to dead load plus imposed load plus wind loading, and the chosen connection has an effective lever arm of 337 mm (dimension A for a 406 × 178 beam from page 67).

Applied panel shear force (for a perimeter column) $= \dfrac{68.0}{0.337}$

$= 201.8$ kN

Panel shear capacity for 203 × 203 × 71 UC $= 353$ kN **OK**

B.10 Serviceability limit state - sway due to wind

Sway deflections can be calculated using any recognised method. The method used in the design example is a simplified procedure developed by Wood and Roberts [12,13].

The actual frame is replaced by a substitute beam-column frame. The basis of the substitute frame is that:

(i) For horizontal loading on the actual frame, the rotations of all joints at any one level are approximately equal, and

(ii) Each beam restrains a column at both ends.

The total stiffness K_b of a beam in the substitute frame is obtained from a summation over all the beams in the actual frame at the level being considered.

The total stiffness K_c of a column in the substitute frame is obtained by a summation over all the columns in the actual frame at the level being considered.

In the simplified method of Wood and Roberts, the sway of a storey is dependent partly on stiffness distribution coefficients calculated for the substitute frame.

To allow for continuity of columns in a multi-storey structure, it is recognised that each floor beam restrains column lengths above and below its own level. This is reflected in the form of the distribution coefficients.

The stiffness distribution coefficients enable a non-dimensional sway index $\bar{n}$ *to be determined from the chart given below (Figure B.11). By definition:*

$$\bar{n} = \frac{)/h}{Fh/(12EK_c)}$$

where $)/h$ *is the sway angle of the storey being considered*
F *is the total wind shear on the column of the substitute frame*
E *is Young's modulus of elasticity (205 kN/mm²)*

Values of k_t *and* k_b *for use with Figure B.11 are defined and calculated in Table B.11.*

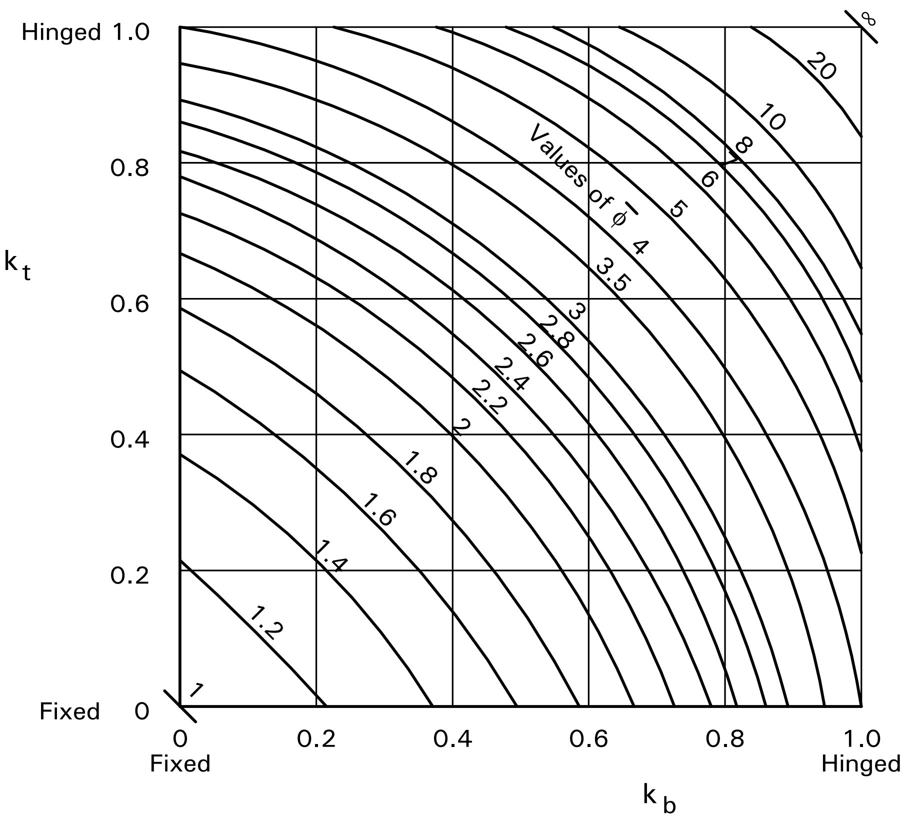

Figure B.11 Sway index ***N***

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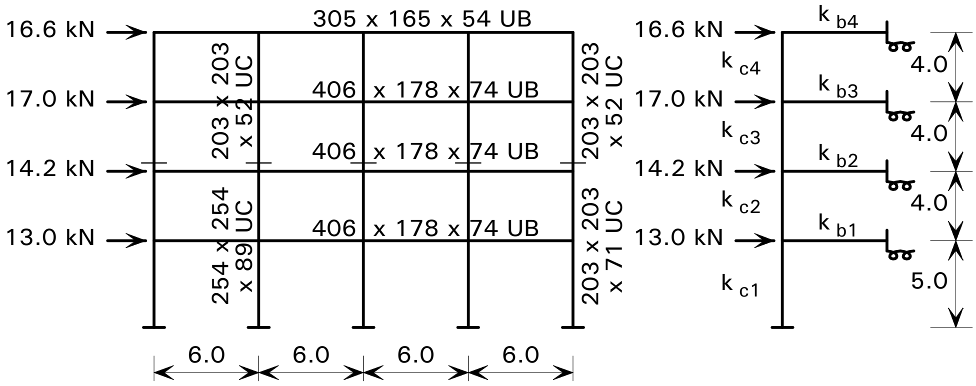


Figure B.12 Frame model for serviceability limit state calculations

Stiffness in substitute frame

Table B.9 Beam stiffness

Storey	I_b (cm^4)	L_b (cm)	$K_b = 3GI_b/L$	K_b (cm^3)
4	11700	600	3 × 4 × 11700/600	234
3	27300	600	3 × 4 × 27300/600	546
2	27300	600	3 × 4 × 27300/600	546
1	27300	600	3 × 4 × 27300/600	546

Table B.10 Column stiffness

Storey	Ext I_c (cm^4)	Int I_c (cm^4)	h (cm)	$K_c = EI_c/h$	K_c (cm^3)
4	5259	5259	400	5 × 5259/400	65.7
3	5259	5259	400	5 × 5259/400	65.7
2	7618	14270	400	(2 × 7618 + 3 × 14270)/400	145
1	7618	14270	500	(2 × 7618 + 3 × 14270)/500	116

__Stiffness distribution coefficients__

Table B.11 Joint stiffness coefficients

Storey	k_t ' $\frac{K_c\%K_u}{K_c\%K_u\%K_{bt}}$	k_t	k_b ' $\frac{K_c\%K_l}{K_c\%K_l\%K_{bb}}$	k_b
4	$\frac{65.7\%0}{65.7\%0\%234}$	0.22	$\frac{65.7\%65.7}{65.7\%65.7\%546}$	0.19
3	$\frac{65.7\%65.7}{65.7\%65.7\%546}$	0.19	$\frac{65.7\%145}{65.7\%145\%546}$	0.28
2	$\frac{145\%65.7}{145\%65.7\%546}$	0.28	$\frac{145\%116}{145\%116\%546}$	0.32
1	$\frac{116\%145}{116\%145\%546}$	0.32	Fixed base	0

where: *K_u is the stiffness of the column above the storey*
K_l is the stiffness of the column below the storey
K_{bt} is the stiffness of the beam above the storey
K_{bt} is the stiffness of the beam below the storey

__Sway deflections__

Table B.12 Sway deflections for a rigid frame

Storey	k_t	k_b	$\bar{N}$	F (kN)	$\frac{)}{h}$ ' $\frac{Fh\ \bar{N}}{12E\ K_c}$	$\frac{)}{h}$	) (mm)
4	0.22	0.19	1.39	16.6	$\frac{16.6 \times 400 \times 1.39}{12 \times 20500 \times 65.8}$	$\frac{1}{1750}$	2.3
3	0.19	0.28	1.47	33.6	$\frac{33.6 \times 400 \times 1.47}{12 \times 20500 \times 65.8}$	$\frac{1}{819}$	4.9
2	0.28	0.32	1.65	47.8	$\frac{47.8 \times 400 \times 1.65}{12 \times 20500 \times 145}$	$\frac{1}{1130}$	3.5
1	0.32	0	1.34	60.8	$\frac{60.8 \times 500 \times 1.34}{12 \times 20500 \times 116}$	$\frac{1}{700}$	7.1
Total						$\frac{1}{955}$	17.8

<u>Allowance for connection flexibility</u>

The deflections calculated treating the frame as rigid-jointed are increased by 50% to make an approximate allowance for connection flexibility (see Section 2.7.2).

The increased deflections shown in Table B.13 are acceptable.

Table B.13 Sway deflections allowing for connection flexibility

Storey	Rigid frame $\left(\frac{\delta}{h}\right)$	Wind-moment frame $\left(1.5\,\frac{\delta}{h}\right)$	Limit	Check
4	1/1750	1/1170	1/300	OK
3	1/819	1/546	1/300	OK
2	1/1130	1/753	1/300	OK
1	1/700	1/467	1/300	OK
Total	1/955	1/637	1/300	OK

APPENDIX C WORKED EXAMPLE: *MINOR AXIS* FRAME

C.1 Introduction

In this example, a *minor axis* wind-moment frame will be designed. The implications for the *major axis* frame considered in Appendix B will also be highlighted.

The following aspects of the frame design are considered:

- framing layout
- loading
- column loads - general
- internal column design
- external column design
- minor axis beams
- major axis beams
- design of minor axis beam to column connections
- design of major axis beam to column connection
- serviceability limit state.

C.1.1 Frame layout

The floor comprises precast concrete units spanning onto the primary beams that frame into the major axis of the columns. The number of bays is four, and the frame spacing is 6 m (see Figure C.1).

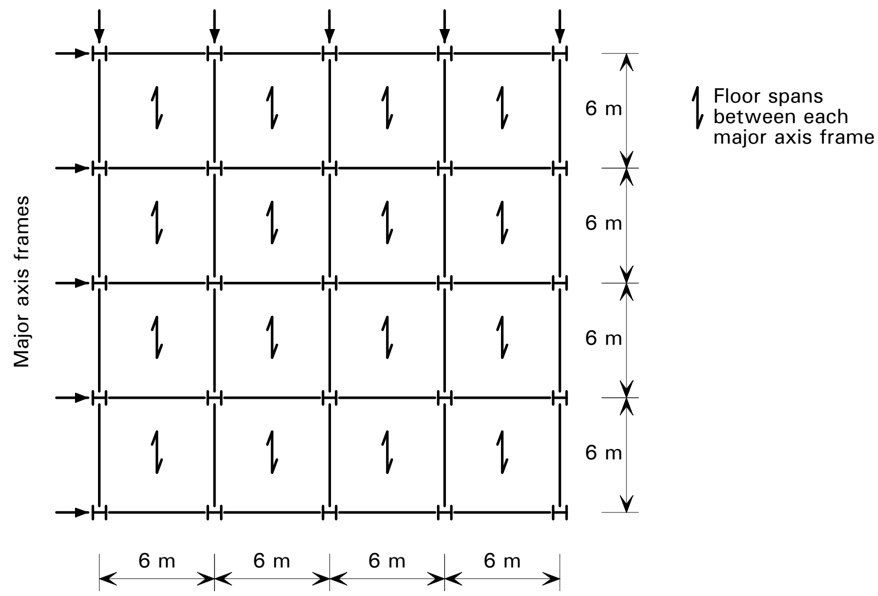


Figure C.1 *Frame layout*

C.1.2 Loading

Vertical loads

The vertical loads (dead and imposed) are as defined in Section B.2. Due to the floor layout, there are no imposed loads on the *minor axis* beams.

Horizontal loads

The horizontal loads are due to wind and notional horizontal forces. The wind loads are assumed to be the same in both directions as the column spacing is the same in both directions. Similarly, the notional horizontal forces are assumed to be the same in both directions. Note also that both the wind loads and the notional horizontal forces need only be applied about one axis at a time, and the most severe case will generally be to apply the loads to the *minor axis* frames.

C.1.3 Column loads - general

The vertical loads and major axis moments on the columns are the same as described in Appendix B.

C.2 Internal column design

C.2.1 Storey 3

Dead load plus imposed load plus notional forces

Design load at ULS

$F_c = 1.4 \times 312 + 1.6 \times 211 = 774\ kN$ — *Table B.7*

Design moment at ULS

Due to notional loads $M_x = 8.03\ kNm$ — *Table B.6*

Similarly $M_y = 8.03\ kNm$

$L = 4\ m$ $L_{Ex} = L_{ey} = 1.5 \times 4 = 6\ m$ — *Section 3.1.1*

Try 203 × 203 × 71 UC S275 steel

Section classification is Class 1 plastic — *Ref 11*

For $L_{Ey} = 6\ m$ $A_g\, p_c = 941\ kN$ — *Ref 11*

For $L = 4\ m$ $M_{bs} = 207\ kNm$ — *Ref 11*

$p_y\, Z_y = 65.2\ kNm$ — *Ref 11*

Moments about the major axis will be due to any eccentric reactions and the 10% restraint moment from the beams. For this example these moments balance and there is therefore no net moment about the major axis.

The notional horizontal forces only act in one direction at a time and the most critical direction will be out of plane.

Overall buckling check

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{774}{941} \% \frac{0}{207} \% \frac{8.03}{65.2} \text{ ' } 0.95 < 1$$

— *BS 5950-1:1990 4.7.7*

Dead load plus imposed load plus wind loading

Design load at ULS

$F_c = 1.2 \times 312 + 1.2 \times 211 = 628\ kN$ — *Table B.7*

Design moment at ULS

Due to wind loads M_x = 1.2 × 16.8 = 20.2 kNm — *Table B.6*

Similarly M_y = 20.2 kNm

Applying the wind load about the minor axis only.

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{628}{941} \% \frac{0}{207} \% \frac{20.2}{65.2} \;' \; 0.98 < 1$$

BS 5950-1:1990 4.7.7

Dead load plus wind loading

F_c = 1.4 × 312 = 437 kN
M_x = 1.4 × 16.8 = 23.5 kN
M_y = 1.4 × 16.8 = 23.5 kN

Applying the wind load about the minor axis only

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{437}{941} \% \frac{0}{207} \% \frac{23.5}{65.2} \;' \; 0.82 < 1$$

BS 5950-1:1990 4.7.7

Use 203 × 203 × 71 UC S275 steel

C.2.2 Storey 1

Dead plus imposed plus notional forces

Design load F_c = 1.4 × 647 + 1.6 × 416 = 1571 kN — *Table B.7*

Design moment M_y *due to notional loads* = 22.9 kNm — *Table B.6*

L = 5 m L_{Ex} = L_{Ey} = 7.5 m — *Section 3.1.1*

Try 305 × 305 × 118 UC S275 steel

Section Class is Class 1 plastic — *Ref 11*

At	L_{Ey} = 7.5 m		$A_g\,p_c$	=	1935 kN	*Ref 11*
	L = 5 m		M_{bs}	=	519 kNm	*Ref 11*
			$p_y\,Z_y$	=	156 kNm	*Ref 11*

$$\frac{F_c}{A_g p_c} \% \frac{M_y}{p_y Z_y} = \frac{1571}{1935} \% \frac{22.9}{156} \ ' \ 0.96 < 1$$

__Dead load plus imposed load plus wind loading__

Design load $F_c = 1.2 \times 647 + 1.2 \times 416 = 1276$ kN — *Table B.7*

Design moment $M_y = 1.2 \times 38.0 = 45.6$ kNm — *Table B.6*
due to wind load

Applying the wind load about the minor axis only

$$\frac{F_c}{A_g p_c} \% \frac{M_y}{p_y Z_y} = \frac{1276}{1935} \% \frac{45.6}{156} \ ' \ 0.95 < 1$$ — *BS 5950-1:1990 4.7.7*

By inspection, dead load plus wind load case will not govern.

__Use 305 × 305 × 118 UC S275 steel__

C.3 External column design

The following calculations are carried out for a non-corner column on the perimeter of the frame.

C.3.1 Storey 3

__Dead load plus imposed load plus notional forces__

Design load $F_c = 1.4 \times 159 + 1.6 \times 105 = 391$ kN — *Table B.8*

__Design moments__

Due to eccentricity (assuming column is 200 mm deep)

$M_x = (1.4 \times 81 + 1.6 \times 90)(0.1 + 0.1) = 51.5$ kNm — *Table B.8*

Due to 10% restraint moment

$M_x = (1.4 \times 12.2 + 1.6 \times 13.5) = \underline{38.7 \text{ kNm}}$

Total $= 90.2$ kNm

Divide moment equally between upper and lower column lengths

M_x = $\underline{45.1 \text{ kNm}}$

Moment due to notional load M_x = 4.0 kNm — *Table B.6*

Moment due to notional horizontal forces M_y = 4.0 kNm

L = 4 m L_{Ex} = L_{Ey} = 1.5 L = 6 m — *Section 3.1.1*

__Try 203 × 203 × 52 UC S275 steel__

Section classification Class 1 plastic — *Ref 11*

For L_E = 6 m $A_g\, p_c$ = 678 kN
For L = 5 m M_{bs} = 150 kNm
$p_y\, Z_y$ = 47.9 kNm

Applying the notional force about the minor axis only

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{391}{678} \% \frac{45.1}{150} \% \frac{4}{47.9} \ ' \ 0.96 \qquad \text{OK}$$

— *BS 5950-1:1990 4.7.7*

__Dead load plus imposed load plus wind loading__

Design load at ULS F_c = 1.2 (159 + 105 + 5.6) = 324 kN — *Tables B.8 and B.2*

Design moment at ULS

Due to eccentricity

M_x = (1.2 × 81 + 1.2 × 90) (0.1 + 0.1) = 41.0 kNm — *Table B.8*

Due to 10% restraint

M_x = (1.2 × 12.2 + 1.2 × 13.5) = $\underline{30.8 \text{ kNm}}$ — *Table B.6*

Total = 71.8 kNm

Divide moment equally between upper and lower column lengths

M_x = $\underline{35.9 \text{ kNm}}$

Moment due to wind M_x = 1.2 × 8.4 = 10.1 kNm — *Table B.6*

The moment about the y-y axis due to wind loads $= 10.1$ *kNm*

Applying the wind load about the minor axis only

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{324}{678} \% \frac{35.9}{150} \% \frac{10.1}{47.9} \text{ ' } 0.93 < 1$$

BS 5950-1:1990 4.7.7

<u>Dead load plus wind loading</u>

Design load at ULS $F_c = 1.4 \times 159 + 1.4 \times 5.6 = 230$ *kN* — *Table B.8 and B.2*

Design moment at ULS

Due to eccentricity

$M_x = 1.4 \times 81\,(0.1 + 0.1) = 22.68$ *kNm*

Due to 10% restraint

$M_x = 1.4 \times 12.2 = \underline{17.1 \text{ kNm}}$ — *Table B.6*

Total $= 39.78$ *kNm*

Divide moment equally between upper and lower column lengths

$M_x = \underline{19.89 \text{ kNm}}$

Moment about y-y axis due to wind $= M_y = 1.4 \times 8.4 = 11.76$ *kNm* — *Table B.6*

Applying the wind load about the minor axis only

$$\frac{F}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{230}{678} \% \frac{19.89}{150} \% \frac{11.76}{47.9} \text{ ' } 0.71 < 1 \qquad \text{OK}$$

BS 5950-1:1990 4.7.7

<u>Use 203 × 203 × 52 UC S275</u>

C.3.2 Storey 1

<u>Dead load plus imposed load plus notional forces</u>

Design load at ULS $F_c = 1.4 \times 332 + 1.6 \times 208 = 798$ *kN* — *Table B.8*

Design moment at ULS

Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 623345
Fax: (01344) 622944

Due to eccentricity (assuming 254 mm deep section)

$M_x \quad = \quad (1.4 \times 81 + 1.6 \times 90)\ (0.125 + 0.1) \quad = \quad 57.9 \text{ kNm}$

Due to 10% restraint

$M_x \quad = \quad (1.4 \times 12.2 + 1.6 \times 13.5) \quad = \quad \underline{38.7 \text{ kNm}}$

$\text{Total} \quad = \quad 96.6 \text{ kNm}$

Divide moment equally between upper and lower column lengths

$M_x \quad = \quad \underline{48.3 \text{ kNm}}$

Moment about y-y axis due to notional load $\quad M_y \quad = \quad 11.5 \text{ kNm}$ *Table B.6*

$L = 5 \text{ m} \qquad L_{Ex} = 1.5\,L = 7.5 \text{ m}$
$\qquad\qquad\quad\;\; L_{Ey} = 1.5\,L = 7.5 \text{ m}$

__Try 254 × 254 × 89 UC S275 steel__

For L_E = 7.5 m	$A_g\, p_c$	=	1170 kN	*Ref 11*
For L = 5 m	M_{bs}	=	316 kNm	*Ref 11*
	$p_y\, Z_y$	=	100 kNm	*Ref 11*

Applying the notional force about the minor axis only

$$\frac{F}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y\, Z_y} = \frac{798}{1170} \% \frac{51.1}{316} \% \frac{11.5}{1009} \;' \; 0.96 < 1 \qquad \text{OK}$$

BS 5950-1:1990 4.7.7

__Dead load plus imposed load plus wind loading__

Design load at ULS $\quad F_c = 1.2\ (332 + 208 + 22.7) = 675 \text{ kN}$ — *Tables B.8 and B.2*

Design moment at ULS

Due to eccentricity

$M_x \quad = \quad 1.2\ (81 + 90)\ (0.1 + 0.125) \quad = \quad 46.17 \text{ kNm}$

Due to 10% restraint

$M_x \quad = \quad 1.2\ (12.2 + 13.5) \quad = \quad \underline{30.8 \text{ kNm}}$

$\text{Total} \quad = \quad 76.97 \text{ kNm}$

Divide moment equally between upper and lower column lengths

$$M_x = \underline{38.5\ kNm}$$

Moment about the y-y axis due to wind loads

$$M_y = 22.8\ kNm$$

Applying the wind load about the minor axis only

$$\frac{F_c}{A_g p_c} \% \frac{M_x}{M_{bs}} \% \frac{M_y}{p_y Z_y} = \frac{675}{1170} \% \frac{38.5}{316} \% \frac{22.8}{100} \ ' \ 0.93 < 1$$ ***OK*** *BS 5950-1:1990 4.7.7*

__Dead load plus wind loading__ *Not critical*

__Use 254 × 254 × 89 UC S275 steel__

C.4 Minor axis beams

The beams framing into the minor axis of the columns will be subjected to relatively little loading. Their size is likely to be determined by the relative stiffness rules given in Section 3.1.2.

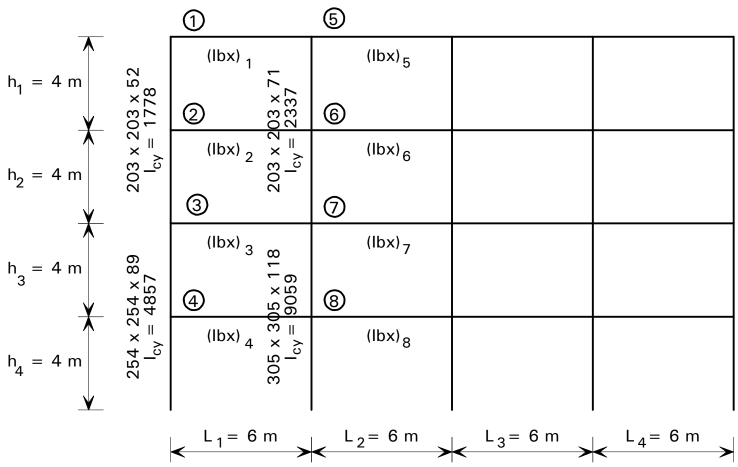


Figure C.2 Column sizes satisfying ULS design requirements

At 1 $(I_{bx})_1 / L_1 \quad > \quad (I_{cy})_1 / h_1$

$(I_{bx})_1 / 6 \quad > \quad 1778/4$

$(I_{bx})_1 \quad > \quad 1778 \times 6/4 = 2667 \text{ cm}^4$

Try 254 × 102 × 22 UB $(I_{bx} = 2841 \text{ cm}^4)$

At 2 $(I_{bx})_2 / L_1 \quad > \quad (I_{cy})_1 / h_1 + (I_{cy})_2 / h_2$

$(I_{bx})_2 / 6 \quad > \quad 1778/4 + 1778/4$

$(I_{bx})_2 \quad > \quad 2 \times 1778 \times 6/4 = 5334 \text{ cm}^4$

Try 305 × 102 × 28 UB $(I_{bx} = 5366 \text{ cm}^4)$

At 3 $(I_{bx})_3 / L_1 \quad > \quad (I_{cy})_2 / h_2 + (I_{cy})_3 / h_3$

$(I_{bx})_3 / 6 \quad > \quad 1778/4 + 4857/4$

$(I_{bx})_3 \quad > \quad (1778 + 4857) \times 6/4 = 9952.5 \text{ cm}^4$

Try 356 × 127 × 39 UB $(I_{bx} = 10170 \text{ cm}^4)$

At 4 $(I_{bx})_4 / L_1 \quad > \quad (I_{cy})_3 / h_3 + (I_{cy})_4 / h_4$

$(I_{bx})_4 / 6 \quad > \quad 4857/4 + 4857/5$

$(I_{bx})_4 \quad > \quad (4857/4 + 4857/5) \times 6 = 13114 \text{ cm}^4$

Try 406 × 140 × 46 UB $(I_{bx} = 15690 \text{ cm}^4)$

Similarly it can be shown that adopting the same section size at each level across the frame will satisfy the remainder of the requirements of Section 3.1.2 for relative stiffness.

Check sizes considering applied loads

Roofs

Design moment at end of beam due to wind loading $1.4 \times 4.16 = 5.8$ kNm — *Table B.3*

Beam unrestrained and unloaded

$L_E = 0.85 \times 6 = 5.1$ m — *BS 5950-1:1990 Table 9*

Use roof beam: 254 × 102 × 22 UB S275 steel

For $n = 1.0 \quad L_E = 5.1$ m $M_b = 16.1$ kNm — *Ref 11*

For $ = -1.0 \quad m = 0.43$ — *BS 5950-1:1990 Table 18*

$\bar{M} = m\,M = 0.43 \times 5.8 = 2.49 \text{ kNm} < 16.1$ OK

3rd Storey

Design moment = 1.4×12.6 = *17.6 kN* — *Table B.3*

Use beam: 305 × 102 × 28 UB S275 steel

L_E = 0.85×6 = *5.1 m*

For $n = 1.0$ L_E = 5.1 M_b = *24.6 kNm* — *Ref 11*

$\bar{M}$ = $m\ M$ = 0.43×17.6 = $7.57 < 24.6$ *kNm* *OK*

2nd Storey

Design moment = 1.4×20.4 = *28.6 kN* — *Table B.3*

Use beam: 356 × 127 × 39 UB S275 steel

L_E = 0.85×6 = *5.1 m*

For $n = 1.0$ L_E = 5.1 M_b = 54.1 — *Ref 11*

$\bar{M}$ = 0.43×28.6 = $12.3 < 54.1$ *OK*

1st Storey

Design moment = 1.4×31 = *43.4 kNm*

Use beam: 406 × 140 × 46 UB S275 steel

L_E 0.85×6 = *5.1 m*

For $n = 1.0$ L_E = 5.1 M_b = 80.4 — *Ref 11*

$\bar{M}$ = 0.43×43.4 = $18.66 < 80.4$ *OK*

C.5 Major axis beams

The beams framing into the major axis of the columns will be designed in the same manner as those for the frame considered in Appendix B.

C.6 Design of minor axis beam to column connections

The design of the minor axis connections will be governed in this case by the wind loading, as there is no other applied load on the minor axis beams.

Roof	$M = 1.4 \times 4.16 = 5.8$ kNm	Shear	$= 5.8/3 =$	1.9 kN	*Table B.3*
3rd Floor	$M = 1.4 \times 12.6 = 17.6$ kNm	Shear	=	5.9 kN	
2nd Floor	$M = 1.4 \times 20.4 = 28.6$ kNm	Shear	=	9.5 kN	
1st Floor	$M = 1.4 \times 31.0 = 43.4$ kNm	Shear	=	14.5 kN	

Connections may be varied at each floor level to suit the beam and column sizes and the required moment capacity.

At the first floor level the connection is required to resist a moment of 43.4 kNm and a shear of 14.5 kN.

Considering the standard connection details given in Appendix D, from page 67 a 12 mm flush end plate with a single row of M20 tension bolts for a 406 × 140 × 46 UB beam has the following capacities:

Moment 69 kNm > 43.4 kNm ***OK***

Shear 258 kN > 14.5 kN ***OK***

The 'column side' of the connection should comprise a 25 mm stiffened plate, detailed according to the guidance given in Section D2.

C.7 Design of major axis beam to column connections

The design of the major axis beam to column connections will be as for the frame braced about the minor axis (see Section B.9).

C.8 Serviceability limit state

Sway due to wind – minor axis

Using the method for plane frames described in Section B.10.

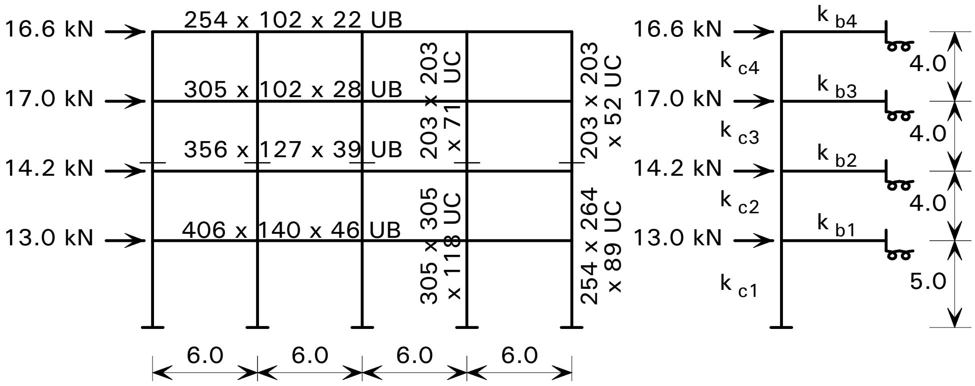


Figure C.3 Substitute frame for SLS check

Table C.1 Beam stiffnesses

Storey	I_b (cm^4)	L (cm)	$K_b = \dfrac{3EI_b}{L}$	K_b (cm^3)
4	2841	600	3 × 4 × 2841/600	56.82
3	5366	600	3 × 4 × 5366/600	107.32
2	10170	600	3 × 4 × 10170/600	203.4
1	15690	600	3 × 4 × 15690/600	313.8

Table C.2 Column stiffnesses

Storey	Ext I_{cy}	Int I_{cy}	h (cm)	$k_c = E\, I_c / h$	K_c (cm^3)
4	1778	2537	400	(2×1778 + 3×2537)/400	27.92
3	1779	2537	400	(2×1778 + 3×2537)/400	27.92
2	4857	9059	400	(2×4857 + 3×9059)/400	92.23
1	4857	9059	500	(2×4857 + 3×9059)/500	73.78

Stiffness distribution coefficients

Table C.3 Joint stiffness coefficients

Storey	$k_t = \frac{K_c + K_u}{K_c + K_u + K_{bt}}$	k_t	$k_b = \frac{K_c + K_l}{K_c + K_l + K_{bb}}$	k_b
4	*(27.9+0)/(27.9+0+56.82)*	*0.33*	*(27.9+27.9)/(27.9+27.9+107.32)*	*0.34*
3	*(27.9+27.9)/(27.9+27.9+107.32)*	*0.34*	*(27.9+92.2)/(27.9+92.2+203.4)*	*0.37*
2	*(92.2+27.9)/(92.2+27.9+203.4)*	*0.37*	*(92.2+73.8)/(92.2+73.8+313.8)*	*0.35*
1	*(73.8+92.2)/(73.8+92.2+313.8)*	*0.35*	*Fixed Base*	*0*

where: K_u *is the stiffness of the column above the storey*
K_l *is the stiffness of the column below the storey*
K_{bt} *is the stiffness of the beam above the storey*
K_{bt} *is the stiffness of the beam below the storey*

Table C.4 Sway deflections for a rigid frame

Storey	k_t	k_b	$\bar{N}$	*F (kN)*	$\frac{\Delta}{h} = \frac{F h N}{12 E k_c}$	$\frac{\Delta}{h}$	Δ_{mm}
4	*0.33*	*0.34*	*1.75*	*16.6*	$\frac{16.6 \times 400 \times 1.75}{12 \times 20500 \times 27.9}$	*1/591*	*6.8*
3	*0.34*	*0.37*	*1.85*	*33.6*	$\frac{33.6 \times 400 \times 1.85}{12 \times 20500 \times 27.9}$	*1/276*	*14.5*
2	*0.37*	*0.35*	*1.85*	*47.8*	$\frac{47.8 \times 400 \times 1.85}{12 \times 20500 \times 92.2}$	*1/641*	*6.2*
1	*0.35*	*0*	*1.35*	*60.8*	$\frac{60.8 \times 500 \times 1.35}{12 \times 20500 \times 73.8}$	*1/442*	*11.3*
Total						*1/438*	*38.8*

Increase the deflections by 50% to allow for connection flexibility.

Table C.5 Sway deflections allow for connection flexibility

Storey	*Rigid frame ()/h)*	*Wind-moment frame (1.5 */h)*	*Limit*	*Check*
4	*1/591*	*1/394*	*1/300*	*OK*
3	*1/276*	*1/184*	*1/300*	*Fails*
2	*1/641*	*1/477*	*1/300*	*OK*
1	*1/442*	*1/295*	*1/300*	*Fails*
Total	*1/438*	*1/292*	*1/300*	*Fails*

From Table C.5 it is clear that the allowable deflections are exceeded at levels 1 and 3.

The stiffness of the frame could be increased by:

1. *Increasing beam sizes*
2. *Increasing column sizes*
3. *Increasing both beam and column sizes.*

The effectiveness of each of these options will depend on the frame in question; for the purposes of this example it was decided to increase the column sizes. The final member sizes are as shown in Figures C.4 and C.5.

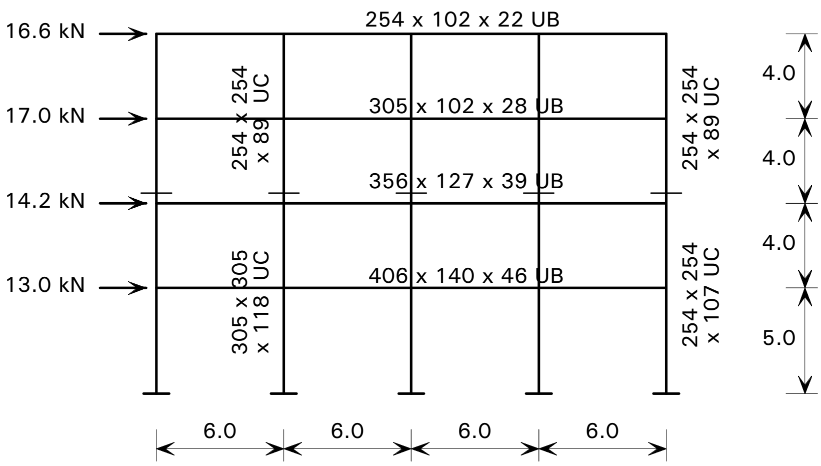


Figure C.4 Column and beam sizes in minor axis direction

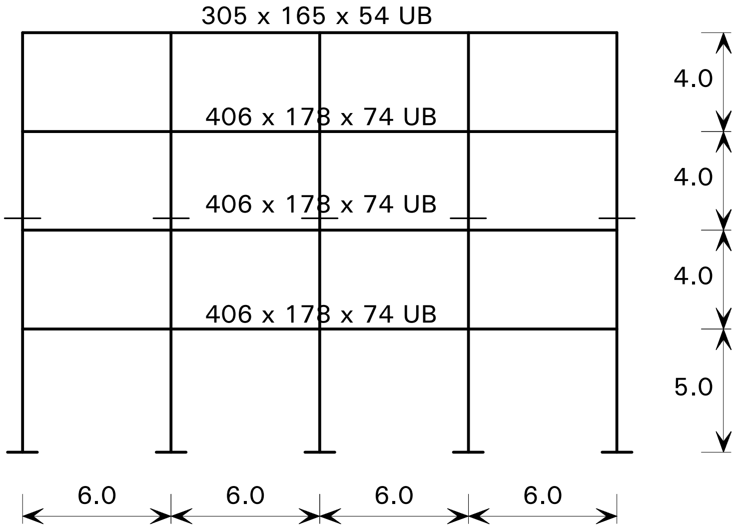


Figure C.5 Beam sizes for major axis frames

APPENDIX D CONNECTION DETAILS AND CAPACITIES

This Appendix covers connections suitable for connecting beams to the major axis of columns. The information given has been taken from *Joints in steel construction: Moment connections* [1], where full design procedures and other background information can be found. The shading that is present in the capacity tables in *Joints in steel construction: Moment connections* [1], which is used to identify sections that are not Class 1, has been omitted in this Appendix. It is not required because the scope of the method has now been broadened to include Class 2 sections (see Section 1.3.2).

D.1 Major axis connections

D.1.1 Notes on use of the tables

In this Appendix, capacity tables are given for connections using M20 8.8 bolts, followed by tables for similar connections with M24 8.8 bolts. All connections adopt either flush or extended end plates that are symmetrical about the centre-line of the beam. A table defining the dimensions for detailing is given on page 81.

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 steel. Local column capacities must be checked as described below.

For the connection to perform 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.

D.1.2 Beam side

Moment capacity

The moment capacity for the beam side of the connection is calculated using the method given in *Joints in steel construction: Moment connections* [1]. 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 S275 steel is used, the beam compression flange capacity is less than GF_r.

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

Dimension A

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.

D.1.3 Column side

Tension zone

A tick **T** in the table indicates that the column flange and web in tension have a greater capacity than the corresponding beam. 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 [1].

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 [1].

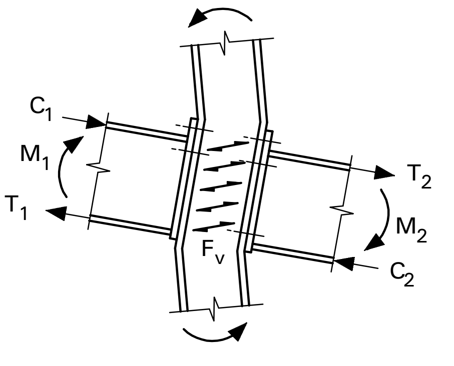
Compression zone

A tick **T** in the table indicates that the column web has a greater compression capacity than the sum of the bolt row forces (EF_r). Note that when the column-side tension zone governs the bolt forces, the stated adequacy of the column compression zone is in relation to these *reduced* bolt values. The check assumes 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 (EF_r). The web must be stiffened to resist EF_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 the same direction, as occurs under wind loading, the panel shear forces from the beams are cumulative (see Figure D.1).



$F_v = C_1 + T_2$

Web panel subject to shear force

Figure D.1 *Forces and deformation of web panel*

D.1.4 Contents of capacity tables

Range of connection types

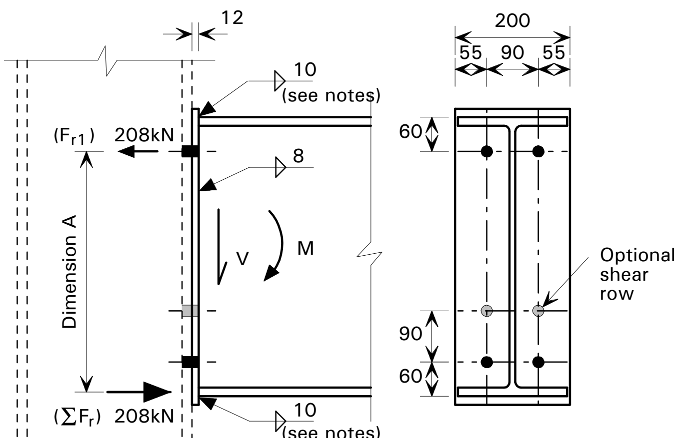
End plate B × T	Type	Bolt (Grade 8.8)	Tension bolt rows	Page
200 × 12	Flush	M20	1	67
200 × 12	Flush	M20	2	68
250 × 12	Flush	M20	2	69
200 × 12	Extended	M20	2	70
250 × 12	Extended	M20	2	71
200 × 12	Extended	M20	3	72
250 × 12	Extended	M20	3	73
200 × 15	Flush	M24	1	74
200 × 15	Flush	M24	2	75
250 × 15	Flush	M24	2	76
200 × 15	Extended	M24	2	77
250 × 15	Extended	M24	2	78
200 × 15	Extended	M24	3	79
250 × 15	Extended	M24	3	80

Dimensions for detailing are shown on page 81.

Beam side

1 row M20 8.8 bolts
200 × 12 S275 flush end plate

Beam serial size	Dimension 'A' (mm)	Moment capacity (kNm)
	Beam - S275 and S355	
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



Vertical shear capacity
258kN without shear row
442kN with shear row

Column side

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

Tension zone:

F_{r1}

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

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

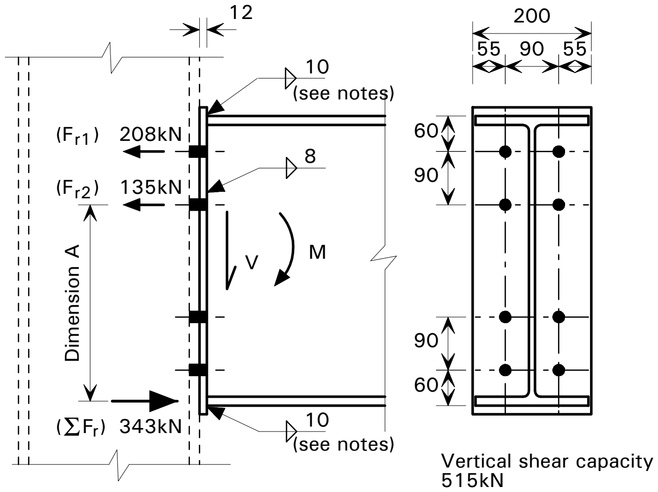
Compression zone:

T Column capacity exceeds EF_r.

Beam side

2 rows M20 8.8 bolts
200 × 12 S275 flush end plate

Beam - S275 and 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



Column side

S275				Column serial size	S355			
Panel shear capacity (kN)	Tension zone F_{r1} (kN)	Tension zone F_{r2} (kN)	Compression zone		Compression zone	Tension zone F_{r1} (kN)	Tension zone F_{r2} (kN)	Panel shear capacity (kN)
1000	**T**	**T**	**T**	356 × 368 × 202	**T**	**T**	**T**	1300
849	**T**	**T**	**T**	177	**T**	**T**	**T**	1110
725	**T**	**T**	**T**	153	**T**	**T**	**T**	944
605	**T**	**T**	**T**	129	**T**	**T**	**T**	788
1037	**T**	**T**	**T**	305 × 305 × 198	**T**	**T**	**T**	1350
816	**T**	**T**	**T**	158	**T**	**T**	**T**	1060
703	**T**	**T**	**T**	137	**T**	**T**	**T**	916
595	**T**	**T**	**T**	118	**T**	**T**	**T**	775
503	**T**	**T**	**T**	97	**T**	**T**	**T**	649
882	**T**	**T**	**T**	254 × 254 × 167	**T**	**T**	**T**	1150
685	**T**	**T**	**T**	132	**T**	**T**	**T**	893
551	**T**	**T**	**T**	107	**T**	**T**	**T**	718
434	**T**	**T**	**T**	89	**T**	**T**	**T**	566
360	**T**	**T**	**T**	73	**T**	**T**	**T**	465
459	**T**	**T**	**T**	203 × 203 × 86	**T**	**T**	**T**	598
353	**T**	**T**	**T**	71	**T**	**T**	**T**	460
322	**T**	**T**	**T**	60	**T**	**T**	**T**	415
272	**T**	**T**	**T**	52	**T**	**T**	**T**	351
245	198	97	**T**	46	**T**	**T**	**T**	316

Tension zone:

F_{r1} F_{r2}

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

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

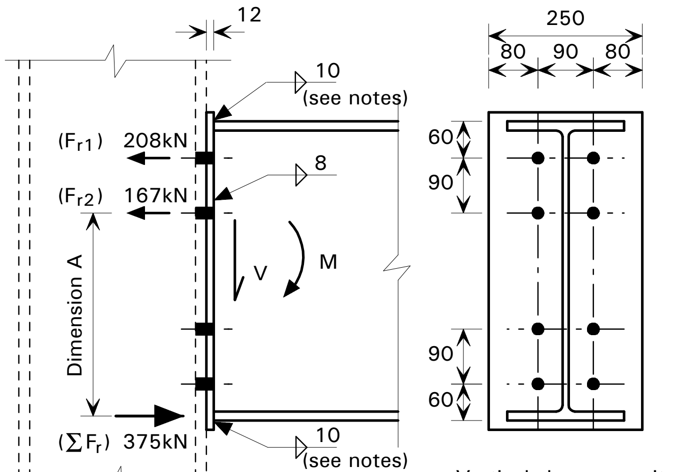
Compression zone:

T Column capacity exceeds EF_r.

Beam side

2 rows M20 8.8 bolts
250 × 12 S275 flush end plate

Beam - S275 and 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



Vertical shear capacity 515kN

Column side

S275					S355			
Panel shear capacity (kN)	**Tension zone F_{r1} (kN)**	**Tension zone F_{r2} (kN)**	**Compression zone**	**Column serial size**	**Compression zone**	**Tension zone F_{r1} (kN)**	**Tension zone F_{r2} (kN)**	**Panel shear capacity (kN)**
1000	T	T	T	356 × 368 × 202	T	T	T	1300
849	T	T	T	177	T	T	T	1110
725	T	T	T	153	T	T	T	944
605	T	T	T	129	T	T	T	788
1037	T	T	T	305 × 305 × 198	T	T	T	1350
816	T	T	T	158	T	T	T	1060
703	T	T	T	137	T	T	T	916
595	T	T	T	118	T	T	T	775
503	T	T	T	97	T	T	T	649
882	T	T	T	254 × 254 × 167	T	T	T	1150
685	T	T	T	132	T	T	T	893
551	T	T	T	107	T	T	T	718
434	T	T	T	89	T	T	T	566
360	T	T	T	73	T	T	T	465
459	T	T	T	203 × 203 × 86	T	T	T	598
353	T	T	T	71	T	T	T	460
322	T	T	T	60	T	T	T	415
272	T	T	S(360)	52	T	T	T	351
245	198	97	T	46	T	T	T	316

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

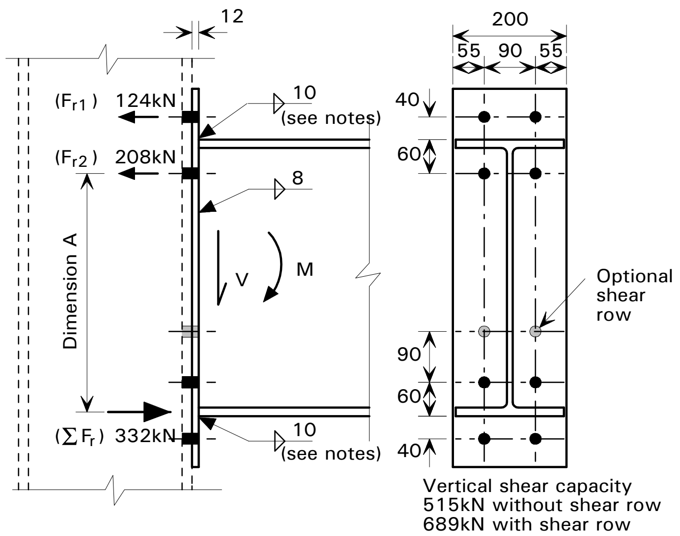
Compression zone:
T Column capacity exceeds EF_r.
S (xxx*)* Column requires stiffening to resist GF_r (value is the column web capacity).

2 rows M20 8.8 bolts
200 × 12 S275 extended end plate

Beam side

Beam - S275 and 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 Panel shear capacity (kN)	S275 Tension zone F_{r1} (kN)	S275 Tension zone F_{r2} (kN)	S275 Compression zone	Column serial size	S355 Compression zone	S355 Tension zone F_{r1} (kN)	S355 Tension zone F_{r2} (kN)	S355 Panel shear capacity (kN)
1000	**T**	**T**	**T**	356 × 368 × 202	**T**	**T**	**T**	1300
849	**T**	**T**	**T**	177	**T**	**T**	**T**	1110
725	**T**	**T**	**T**	153	**T**	**T**	**T**	944
605	**T**	**T**	**T**	129	**T**	**T**	**T**	788
1037	**T**	**T**	**T**	305 × 305 × 198	**T**	**T**	**T**	1350
816	**T**	**T**	**T**	158	**T**	**T**	**T**	1060
703	**T**	**T**	**T**	137	**T**	**T**	**T**	916
595	**T**	**T**	**T**	118	**T**	**T**	**T**	775
503	**T**	**T**	**T**	97	**T**	**T**	**T**	649
882	**T**	**T**	**T**	254 × 254 × 167	**T**	**T**	**T**	1150
685	**T**	**T**	**T**	132	**T**	**T**	**T**	893
551	**T**	**T**	**T**	107	**T**	**T**	**T**	718
434	**T**	**T**	**T**	89	**T**	**T**	**T**	566
360	**T**	206	**T**	73	**T**	**T**	**T**	465
459	**T**	**T**	**T**	203 × 203 × 86	**T**	**T**	**T**	598
353	**T**	**T**	**T**	71	**T**	**T**	**T**	460
322	**T**	191	**T**	60	**T**	**T**	202	415
272	**T**	181	**T**	52	**T**	**T**	190	351
245	**T**	107	**T**	46	**T**	**T**	181	316

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

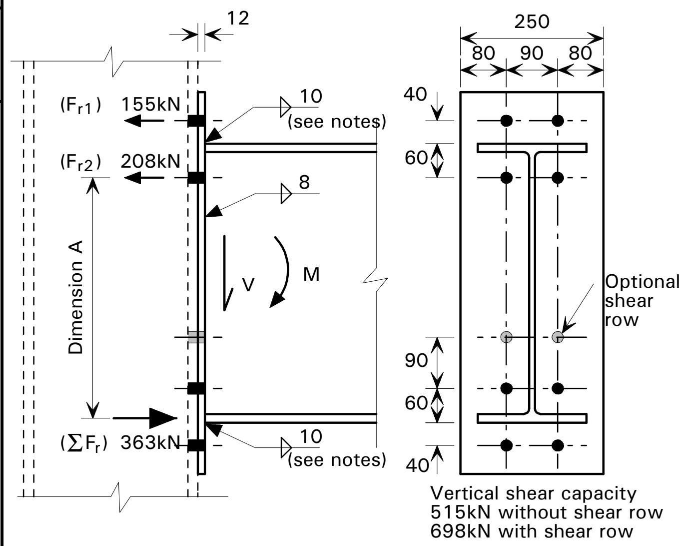
Compression zone:
T Column capacity exceeds EF_r.

Beam side

2 rows M20 8.8 bolts
250 × 12 S275 extended end plate

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

Tension zone:

F_{r1} F_{r2}

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

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

Compression zone:

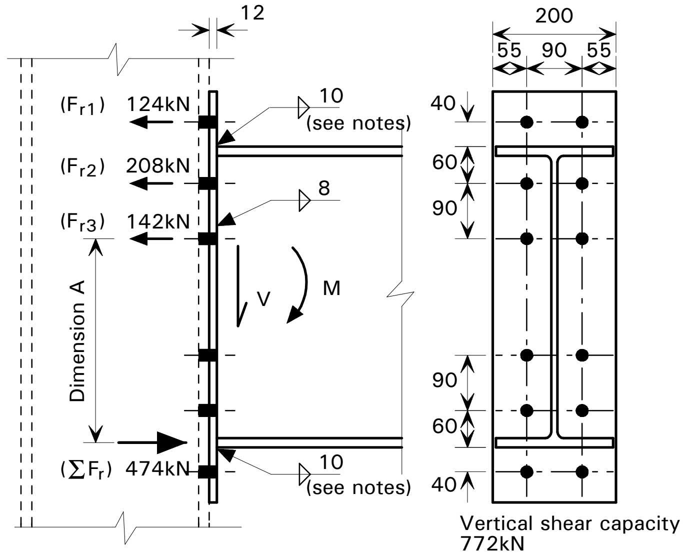
T Column capacity exceeds EF_r.

Beam side

3 rows M20 8.8 bolts
200 × 12 S275 extended end plate

Beam serial size	Dimension 'A' (mm)	Moment capacity (kNm)
Beam - S275 and S355		
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



Vertical shear capacity 772kN

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 Compression zone	Column serial size	S355 Compression 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	**T**	**T**	**T**	**T**	356 × 368 × 202	**T**	**T**	**T**	**T**	1300
849	**T**	**T**	**T**	**T**	177	**T**	**T**	**T**	**T**	1110
725	**T**	**T**	**T**	**T**	153	**T**	**T**	**T**	**T**	944
605	**T**	**T**	**T**	**T**	129	**T**	**T**	**T**	**T**	788
1037	**T**	**T**	**T**	**T**	305 × 305 × 198	**T**	**T**	**T**	**T**	1350
816	**T**	**T**	**T**	**T**	158	**T**	**T**	**T**	**T**	1060
703	**T**	**T**	**T**	**T**	137	**T**	**T**	**T**	**T**	916
595	**T**	**T**	**T**	**T**	118	**T**	**T**	**T**	**T**	775
503	**T**	**T**	**T**	**T**	97	**T**	**T**	**T**	**T**	649
882	**T**	**T**	**T**	**T**	254 × 254 × 167	**T**	**T**	**T**	**T**	1150
685	**T**	**T**	**T**	**T**	132	**T**	**T**	**T**	**T**	893
551	**T**	**T**	**T**	**T**	107	**T**	**T**	**T**	**T**	718
434	**T**	**T**	**T**	**T**	89	**T**	**T**	**T**	**T**	566
360	**T**	206	**T**	*S (436)*	73	**T**	**T**	**T**	**T**	465
459	**T**	**T**	**T**	**T**	203 × 203 × 86	**T**	**T**	**T**	**T**	598
353	**T**	**T**	**T**	**T**	71	**T**	**T**	**T**	**T**	460
322	**T**	191	**T**	*S (440)*	60	**T**	**T**	202	**T**	415
272	**T**	181	121	*S (360)*	52	**T**	**T**	190	**T**	351
245	**T**	107	90	*S (313)*	46	*S (404)*	**T**	181	118	316

Tension zone:

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

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

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

Compression zone:

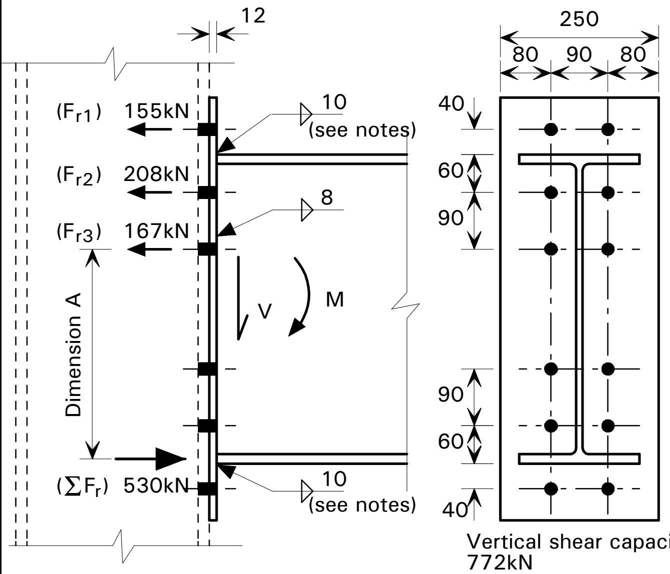
T Column capacity exceeds EF_r.

S (xxx) Column requires stiffening to resist GF_r (value is the column web capacity).

Beam side

3 rows M20 8.8 bolts
250 × 12 S275 extended end plate

Beam - S275 and 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



Vertical shear capacity
772kN

Column side

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

Tension zone:

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

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

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

Compression zone:

T Column capacity exceeds EF_r.

S (xxx*)* Column requires stiffening to resist GF_r (value is the column web capacity).

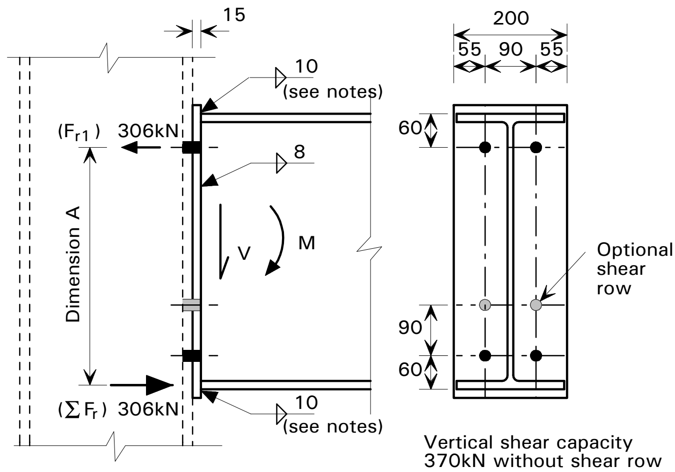
Beam side

1 row M24 8.8 bolts
200 × 15 S275 flush end plate

Beam - S275 and 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



Vertical shear capacity
370kN without shear row
634kN with shear row

Column side

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

Tension zone:
F_{r1}
T 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:

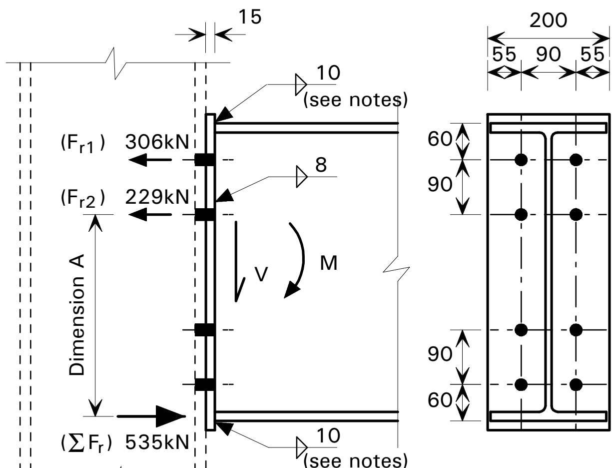
T Column capacity exceeds EF_r.

2 rows M24 8.8 bolts
200 × 15 S275 flush end plate

Beam side

Beam - S275 and 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



Vertical shear capacity 739kN

Column side

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

Tension zone:

F_{r1} F_{r2}

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

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

Compression zone:

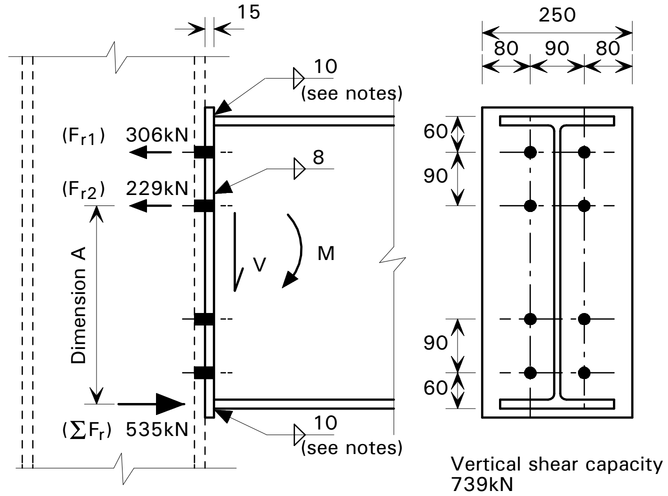
T Column capacity exceeds EF_r.

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

Beam side

2 rows M24 8.8 bolts
250 × 15 S275 flush end plate

Beam - S275 and 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



Vertical shear capacity 739kN

Column side

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

Tension zone:

F_{r1} F_{r2}

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

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

Compression zone:

T Column capacity exceeds EF_r.

S (xxx) Column reduced stiffening to resist EF_r (value is the column web capacity).

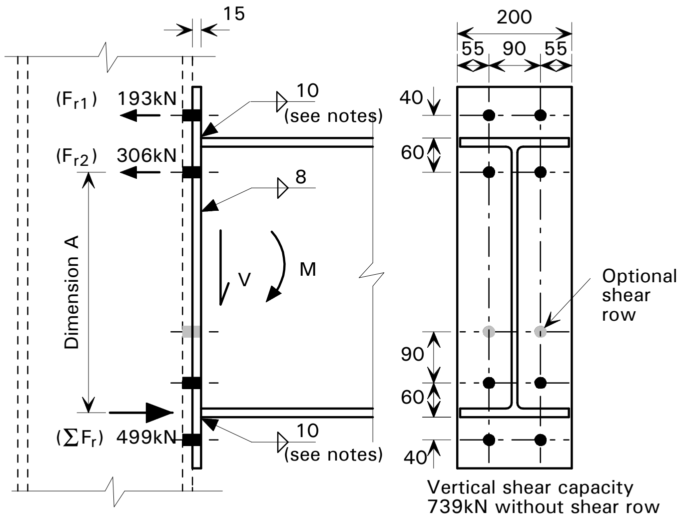
Beam side

2 rows M24 8.8 bolts
200 × 15 S275 extended end plate

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

Beam - S275 and S355

*406 × 140 × 39 These sections
356 × 127 × 33 suitable in S355 only



Vertical shear capacity
739kN without shear row
1003kN with shear row

Column side

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

Tension zone:

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

Compression zone:

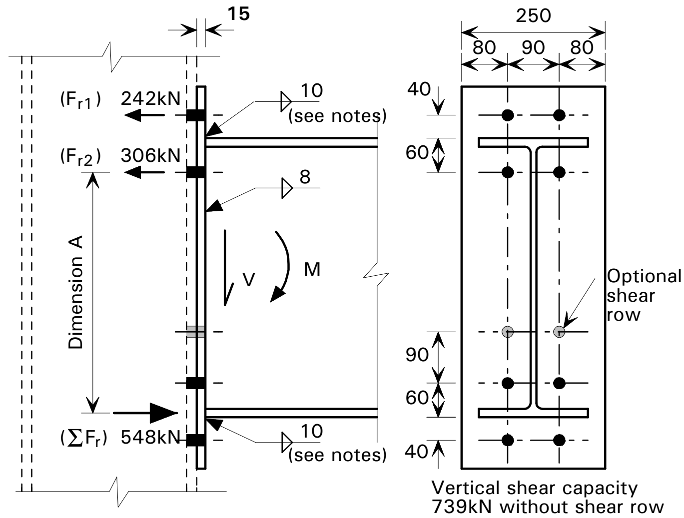
T Column capacity exceeds EF_r.
S (xxx) Column reduced stiffening to resist EF_r (value is the column web capacity).

2 rows M24 8.8 bolts
250 × 15 S275 extended end plate

Beam side

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
	294	195

Beam - S275 and S355



Vertical shear capacity
739kN without shear row
1003kN with shear row

Column side

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

Tension zone:

F_{r1} F_{r2}

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

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

Compression zone:

T Column capacity exceeds EF_r.

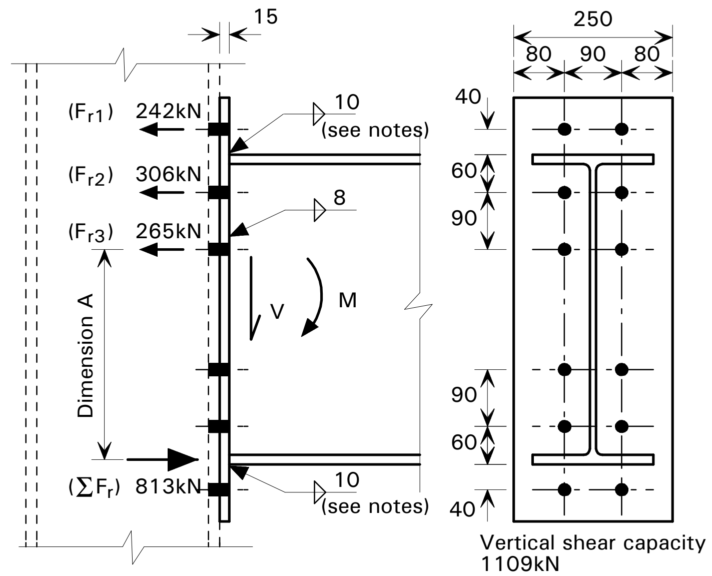
S (xxx) Column requires stiffening to resist EF_r (value is the column web capacity).

Beam side

3 rows M24 8.8 bolts
200 × 15 S275 extended end plate

Beam - S275 and S355

Beam serial size	Dimension 'A' (mm)	Moment capacity (kNm)
533 × 210	372	342
457 × 191	297	286



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 Compression zone	Column serial size	S355 Compression 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	**T**	**T**	**T**	**T**	356 × 368 × 202	**T**	**T**	**T**	**T**	1300
849	**T**	**T**	**T**	**T**	177	**T**	**T**	**T**	**T**	1110
725	**T**	**T**	**T**	**T**	153	**T**	**T**	**T**	**T**	944
605	**T**	**T**	**T**	S *(605)*	129	**T**	**T**	**T**	**T**	788
1037	**T**	**T**	**T**	**T**	305 × 305 × 198	**T**	**T**	**T**	**T**	1350
816	**T**	**T**	**T**	**T**	158	**T**	**T**	**T**	**T**	1060
703	**T**	**T**	**T**	**T**	137	**T**	**T**	**T**	**T**	916
595	**T**	**T**	**T**	S *(692)*	118	**T**	**T**	**T**	**T**	775
503	**T**	**T**	**T**	S *(553)*	97	S *(713)*	**T**	**T**	**T**	649
882	**T**	**T**	**T**	**T**	254 × 254 × 167	**T**	**T**	**T**	**T**	1150
685	**T**	**T**	**T**	**T**	132	**T**	**T**	**T**	**T**	893
551	**T**	**T**	**T**	**T**	107	**T**	**T**	**T**	**T**	718
434	**T**	301	**T**	S *(557)*	89	S *(725)*	**T**	**T**	**T**	566
360	**T**	274	**T**	S *(436)*	73	S *(563)*	**T**	289	**T**	465
459	**T**	**T**	**T**	S *(701)*	203 × 203 × 86	**T**	**T**	**T**	**T**	598
353	**T**	276	**T**	S *(512)*	71	S *(666)*	**T**	293	**T**	460
322	**T**	221	**T**	S *(440)*	60	S *(568)*	**T**	269	**T**	415
272	**T**	131	118	S *(360)*	52	S *(464)*	**T**	215	152	351
245	**T**	100	90	S *(313)*	46	S *(404)*	**T**	129	116	316

Tension zone:

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

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

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

Compression zone:

T Column capacity exceeds EF_r.

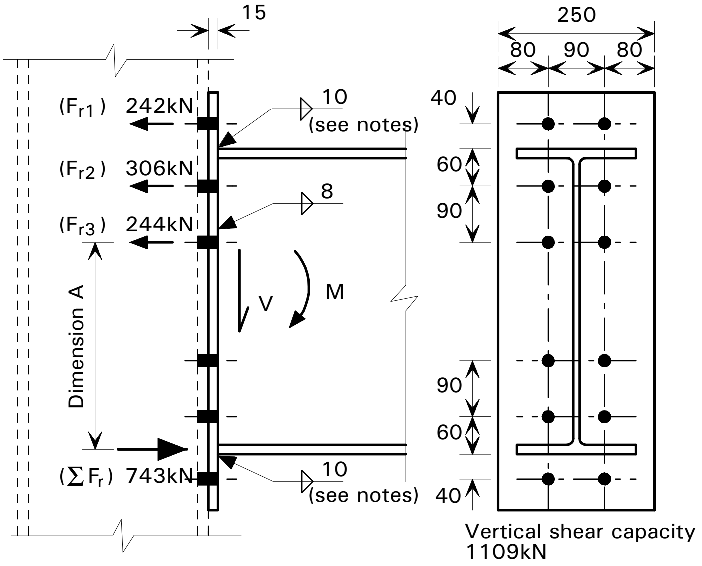
S (xxx) Column requires stiffening to resist EF_r (value is the column web capacity).

Beam side

3 rows M24 8.8 bolts
250 × 15 S275 extended end plate

Beam - S275 and 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 Compression zone	Column serial size	S355 Compression 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	**T**	**T**	**T**	**T**	356 × 368 × 202	**T**	**T**	**T**	**T**	1300
849	**T**	**T**	**T**	**T**	177	**T**	**T**	**T**	**T**	1110
725	**T**	**T**	**T**	S *(766)*	153	**T**	**T**	**T**	**T**	944
605	**T**	**T**	**T**	S *(605)*	129	S *(788)*	**T**	**T**	**T**	788
1037	**T**	**T**	**T**	**T**	305 × 305 × 198	**T**	**T**	**T**	**T**	1350
816	**T**	**T**	**T**	**T**	158	**T**	**T**	**T**	**T**	1060
703	**T**	**T**	**T**	**T**	137	**T**	**T**	**T**	**T**	916
595	**T**	**T**	**T**	S *(692)*	118	**T**	**T**	**T**	**T**	775
503	**T**	**T**	**T**	S *(553)*	97	S *(713)*	**T**	**T**	**T**	649
882	**T**	**T**	**T**	**T**	254 × 254 × 167	**T**	**T**	**T**	**T**	1150
685	**T**	**T**	**T**	**T**	132	**T**	**T**	**T**	**T**	893
551	**T**	**T**	**T**	S *(744)*	107	**T**	**T**	**T**	**T**	718
434	**T**	301	**T**	S *(557)*	89	S *(725)*	**T**	**T**	**T**	566
360	**T**	274	182	S *(436)*	73	S *(563)*	**T**	289	**T**	465
459	**T**	**T**	**T**	S *(701)*	203 × 203 × 86	**T**	**T**	**T**	**T**	598
353	**T**	276	**T**	S *(512)*	71	S *(666)*	**T**	293	**T**	460
322	**T**	221	155	S *(440)*	60	S *(568)*	**T**	269	264	415
272	**T**	131	118	S *(360)*	52	S *(464)*	**T**	215	152	351
245	204	100	90	S *(313)*	46	S *(404)*	**T**	129	116	316

Tension zone:

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

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

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

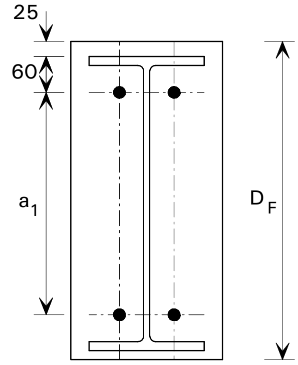
Compression zone:

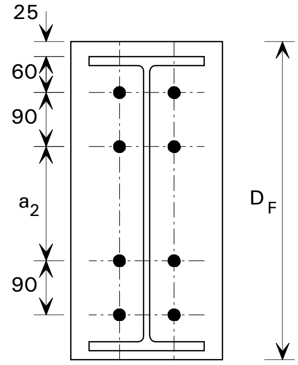
T Column capacity exceeds EF_r.

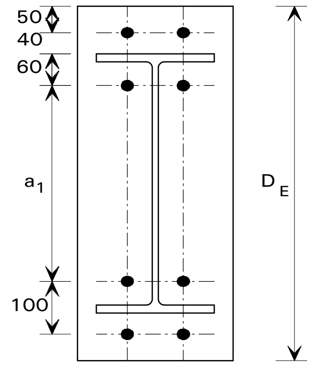
S (xxx) Column requires stiffening to resist EF_r (value is the column web capacity).

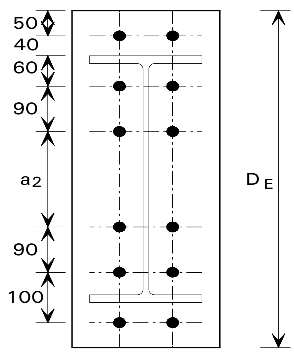
Standard connections - dimensions for detailing

	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	575	395	750	880
152	570	390		
140	565	385		
125	560	380		
610 × 229 × 140	500	320	670	800
125	490	310		
113	490	310		
101	480	300		
533 × 210 × 122	425	245	600	730
109	420	240		
101	415	235		
92	415	235		
82	410	230		
457 × 191 × 98	350	170	520	650
89	345	165		
82	340	160		
74	340	160		
67	335	155		
457 × 152 × 82	345	165	520	650
74	340	160		
67	340	160		
60	335	155		
52	330	150		
406 × 178 × 74	295	115	300	600
67	290	110		
60	285	105		
54	285	105		
406 × 140 × 46	280	100	450	580
39	275	95		
356 × 171 × 67	245		420	550
57	240			
51	235			
45	230			
356 × 127 × 39	235		410	540
33	230			
305 × 165 × 54	190		360	490
46	185			
40	185			
305 × 127 × 48	190		360	490
42	185			
37	185			
305 × 102 × 33	195		370	500
28	190			
25	185			
254 × 146 × 43	140		310	440
37	135			
31	135			
254 × 102 × 28	140		310	440
25	135			
22	135			









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

SCI-P263

Typeset and page make-up by The Steel Construction Institute, Ascot SL5 7QN
Printed and bound by Thanet Press, Margate CT9 1NU
1500 12/99 BCB766