The Institution of Structural Engineers
The Institution of Civil Engineers

November 1989

Manual for the design of steelwork building structures

Published by the Institution of Structural Engineers

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Constitution of *ad hoc* Committee

B. H. Fisher, BSc, CEng, FIStructE, FICE, *Chairman*
P. G. Cobb, CEng, MICE
J. D. Hannon, CEng, FIStructE, FICE
S. E. Harridge, BSc, MSc, DIC, CEng, FIStructE, MICE
S. Higginbottom, CEng, MIStructE
D. M. Howell, BEng, CEng, MICE
W. F. Morgan, CEng, FIStructE
R. S. Narayanan, BE(Hons), MSc, DIC, CEng, FIStructE
R. F. D. Povey, CEng, FIStructE
R. Stainsby, DIC, CEng, FIStructE
R. W. Turner, CEng, FIStructE, FICE
A. D. Weller, CEng, FIStructE, MBCS
G. Buckton, CEng, FIStructE, MWeldI, *Corresponding member*
R. J. W. Milne, BSc, *Secretary*

Contents

Foreword

In 1986 the Institution of Structural Engineers formed a Committee to prepare a *Manual* for the design of structural steelwork which would be compatible with BS 5950, which was published in 1985. The Institution of Civil Engineers has joined in this task and this document is the result. It has been written by and for practising designers and thus reflects the logical sequence of operations that a designer follows.

The *Manual* covers the majority of multistorey and single-storey buildings, but with the deliberate exclusion of some items. For example, plate girders and crane gantries are not covered and the range of multistorey structures is limited to those not dependent on the bending of columns for resistance against horizontal forces. This limitation recognizes that buildings are usually designed to be braced by strongpoints such as shear walls, infill panels and the like.

The Committee has aimed at clarity and logical presentation of structural steelwork design practice in writing the *Manual* which offers practical guidance on how to design safe, robust and durable structures. It is hoped that the concise format will be welcomed.

The preparation of the *Manual* has proceeded concurrently with, but independently of, the preparation of amendment no. 1 to BS 5950. Helpful comment has been received from members of the BS 5950 Committee, and from many members of staff of the Steel Construction Institute. The Institutions and I are indeed grateful for the many helpful comments on the penultimate draft of the *Manual* received from SCI.

Users will note that the recommendations given in this *Manual* fall within the wider range of options in BS 5950, and the amendment no. 1 to BS 5950 which it is anticipated will be published by BSI by the end of 1989.

During the preparation many people have commented, and I would be grateful if any further comment could be forwarded to the Institution.

Lastly I would like to express my thanks to the members of the Committee and their organizations and also to our Secretary, Mr. R. J. W. Milne, for the enthusiasm and harmonious relations which have characterized our work.

B H FISHER
Chairman

1 Introduction

1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of single and multistorey building structures using structural steelwork. Structures designed in accordance with this manual will normally comply with BS 5950[1] and the anticipated amendment no. 1 to BS 5950.

1.2 Scope of the *Manual*

The range of the structures covered by the *Manual* are:

- braced multistorey structures that do not rely on bending resistance of columns for their overall stability
- single-storey structures using portal frames, posts and lattice trusses or posts and pitched roof trusses.

For structures outside this scope, BS 5950[1] should be used.

1.3 Contents of the *Manual*

The *Manual* covers the following:

- guidance on structural form, framing and bracing including advice on the selection of floors, roofing and cladding systems, and advice on fire and corrosion protection
- step-by-step procedures for designing the different types of structure and structural elements including verification of robustness and design of connections.

1.4 General format of the *Manual*

In the design of structural steelwork it is not practical to include all the information necessary for section design within the covers of one book. Section properties and capacities have been included in the *Manual* when appropriate, but nevertheless reference will frequently need to be made to the *Steelwork design guide to BS 5950: Part 1 1985*, Volume 1[2] (the 'blue book') published by Constrado.

2 General principles

This Section outlines the general principles that apply to the design of structural steel buildings.

2.1 General

One engineer should be responsible for the overall design, including stability, so that the design of all structural parts and components is compatible even where some or all of the design and details of parts and components are not made by the same engineer.

The structure should be so arranged that it transmits dead, wind and imposed loads in a direct manner to the foundations. The general arrangement should lead to a robust and stable structure that will not overturn or collapse progressively under the effects of misuse or accidental damage to any one element. Consideration should also be given to the erection procedure and stability during construction.

2.2 Stability

2.2.1 Multistorey braced structures

Lateral stability in two directions approximately at right-angles to each other should be provided by a system of vertical and horizontal bracing within the structure so that the columns will not be subject to sway moments. Bracing can generally be provided in the walls enclosing the stairs, lifts, service ducts, etc. Additional stiffness can also be provided by bracing within other external or internal walls. The bracing should preferably be distributed throughout the structure so that the combined shear centre is located approximately on the line of the resultant on plan of the applied overturning forces. Where this is not possible, torsional moments may result, which must be considered when calculating the load carried by each braced bay.

Braced bays should be effective throughout the full height of the building. If it is essential for bracing to be discontinuous at one level, provision must be made to transfer the forces to other braced bays.

2.2.2 Single-storey structures

Lateral stability to these structures should be provided in two directions approximately at right angles to each other. This may be achieved by:

- rigid framing, or
- vertical braced bays in conjunction with plan bracing.

2.2.3 Forms of bracing

Bracing may consist of any of the following:

- horizontal bracing
  - triangulated steel members
  - concrete floors or roofs
  - adequately designed and fixed profiled steel decking
- vertical bracing
  - triangulated steel members
  - reinforced concrete walls preferably not less than 180 mm in thickness.
  - masonry walls preferably not less than 150mm in thickness adequately pinned and tied to the steel frames. Precautions should be taken to prevent such walls being removed at a later stage, and temporary bracing provided during erection before such masonry walls are constructed.

2.3 Robustness

All members of a structure should be effectively tied together in the longitudinal, transverse and vertical directions as set out in Sections 7 and 9. Members whose failure would cause collapse of more than a limited part of the structure adjacent to them should be avoided. Where this is not possible, alternative load paths should be identified or the member in question strengthened.

2.4 Movement joints

Joints should be provided to minimize the effects of movements arising from temperature variations and settlement. The effectiveness of movement joints depends on their location, which should divide the structure into a number of individual sections. The joints should pass through the whole structure above ground level in one plane. The structure should be framed on both sides of the joint, and each section should be structurally independent and designed to be stable and robust without relying on the stability of adjacent sections.

Joints may also be required where there is a significant change in the type of foundation, plan configuration or the height of the structure. Where detailed calculations are not made, joints to permit movement of 15 to 25mm should normally be provided at approximately 50m centres both longitudinally and transversely. For single-storey sheeted buildings it may be appropriate to increase these spacings. Attention should be drawn to the necessity of incorporating joints in the finishes and in the cladding at the movement joint locations.

In addition a gap should generally be provided between steelwork and masonry cladding to allow for the movement of columns under loading.

2.5 Loading

This *Manual* adopts the limit-state principle and the load factor format of BS 5950. The unfactored loads to be used in calculations are obtained as follows:

(*a*) unfactored dead load, G_k: the weight of the structure complete with finishes, fixtures and fixed partitions (BS 648[3]).
(*b*) unfactored imposed load, Q_k (BS 6399, Parts 1 and 3[4]).
(*c*) unfactored wind load, W_k (CP 3, Chapter V, Part 2[5] or BS 6399 Part 2[4], in preparation).
(*d*) notional horizontal load N_k at each level which should be the greater of:

$$1\% \times 1{\cdot}4\ G_k \text{ or } 0{\cdot}5\% \times (1{\cdot}4\ G_k + 1{\cdot}6\ Q_k)$$

where G_k and Q_k are the unfactored loads from the level considered.

2.6 Limit states

2.6.1 Strength and stability limit states

The load combinations and load factors to be used in design for the limit states of strength and stability are shown in Table 1. The factored loads to be used for each load combination should be obtained by multiplying the unfactored loads by the appropriate load factor γ_f from Table 1.

Table 1 Load combinations and load factors γ_f

load combination	load type					
	dead, G_k		imposed, Q_k		wind, W_k	notional, N_k
	adverse	beneficial	adverse	beneficial		
1 dead + imposed	1·4	1·0	1·6	0		
2 dead + wind	1·4	1·0			1·4	
3 dead + wind + imposed	1·2	1·0	1·2	0	1·2	
4 dead + imposed + notional horizontal	1·4		1·6			1·0

The 'adverse' and 'beneficial' factors should be used so as to produce the most onerous condition. When appropriate, temperature effects should be considered with load combinations 1, 2 and 3.

2.6.2 Serviceability limit states

2.6.2.1 Deflection

The structure and its members should be checked for deflections under unfactored imposed loads and unfactored wind loads. The deflections should also be checked where necessary for unfactored dead load + 80% of the unfactored imposed and wind loads.

The deflections for beams arising from unfactored imposed loads should normally be limited to the following values:

cantilevers	length/180
beams carrying plaster or other brittle finish	span/360
all other beams	span/200 and/or that due to check for frequency response

The deflection of columns arising from unfactored imposed and wind loads should normally be limited to the following values:

columns in all single-storey buildings	height/300
columns in multistorey buildings	height of storey/300

For some buildings other values than those shown above may be more appropriate. In particular for multistorey buildings a ratio of height of storey/500 may be more suitable where the cladding cannot accommodate larger movements.

2.6.2.2 Fire resistance

Structural steel members generally require to be protected by insulating materials to enable them to carry their loads during and after a fire. The type and thickness of insulation to be applied depends on the period of fire resistance required, which in turn depends on the use and size of the building; alternatively, fire engineering methods may be used. BS 5950: Part 8[6] (in preparation) may also be consulted.

2.6.2.3 Corrosion protection
Structural steel members often require to be protected against corrosion. The degree of protection required depends on the expected life to the first maintenance, the environment, the degree of exposure, and on the extent to which maintenance is likely to be practicable or possible.

2.6.2.4 Vibration
Vibrations can occur in buildings causing discomfort or structural distress. For simply supported beams this may be minimized by limiting the unfactored dead load deflection to 12mm. Reference may also be made to the SCI *Design guide for vibration of floors*[7]. Floors supporting sensitive equipment or subject to dancing etc. may need special consideration.

2.7 Material properties

2.7.1 Partial factor for materials
The partial factor γ_m for steel to 4360:1986[8] is taken as 1·0.

2.7.2 Design strength p_y
This *Manual* covers the design of structures fabricated from steels supplied to BS 4360, and the design strengths, p_y, should be obtained from Table 2.

Table 2 Design strengths, p_y

BS 4360 : 1986 grade	thickness less than or equal to, mm	sections, plates hollow sections, p_y, N/mm^2
43	16	275
	40	265
	63	255
	100	245
50	16	355
	40	345
	63	340
	100	325

Other steels may be used provided that their design strengths are obtained in a similar manner as in BS 4360.

2.7.3 Brittle fracture
In locations subject to tensile stresses (caused by axial loads or bending moments), brittle fracture should be considered. In general it will be sufficient to limit the thickness of parts to the values shown in Table 3. For conditions not covered in Table 3, reference should be made to BS 5950.

2.7.4 Modulus of elasticity
The modulus of elasticity, E, should be taken as 205 kN/mm^2.

2.7.5 Coefficient of linear expansion
The coefficient of linear expansion, α, should be taken as 12×10^{-6} per °C.

Table 3 Limits to thickness to avoid brittle fracture (sections other than hollow sections)

steel grade	internal (service temperature < – 5°C) mm	external (service temperature < – 15°C) mm
43A	25*	15*
43B	30	20
43C	60	40
50A	20	12
50B	25	16
50C	45	30

*These limiting values do not apply to baseplates designed in accordance with clause 15.1.2; however the values may be increased to 50mm when baseplates transmit moments.

Notes to Table 3

1 The values given apply when the service stress on the component exceeds $100N/mm^2$ *and* the material is at a welded location or unreamed punched holes. For other combinations of service stress and material location, the values of the limits in the Table can be doubled except for grade 43C for which the thickness should be limited to 100mm.

2 The Table does not apply to hollow sections, which may be used without consideration of brittle fracture provided that the thickness of the walls of RHS do not exceed 16mm and those of CHS do not exceed 40mm.

3 For guidance on limiting thickness of plates, wide flats, round and square bars reference should be made to BS 5950.

4 For grades 43B and 50B, option B on page 39 of BS 4360 should be invoked when ordering.

3 Braced multistorey buildings — general

3.1 Introduction

This Section offers advice on the general principles to be applied when preparing a scheme for a braced multistorey structure. The aim should be to establish a structural scheme that is practicable, sensibly economic, and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Loads should be carried to the foundation by the shortest and most direct routes. In constructional terms, simplicity implies (among other matters) repetition, avoidance of congested, awkward or structurally sensitive details, with straightforward temporary works and minimal requirements for unorthodox sequencing to achieve the intended behaviour of the completed structure.

Sizing of structural members should be based on the longest spans (slabs and beams) and largest areas of roof and/or floors carried (beams, columns, walls and foundations). The same sections should be assumed for similar but less onerous cases — this saves design and costing time and is of actual advantage in producing visual and constructional repetition and hence, ultimately, cost benefits.

Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good 'benchmark'. Scheme drawings should be prepared for discussion and budgeting purposes incorporating such items as general arrangement of the structure including, bracing, type of floor construction, critical and typical beam and column sizes, and typical edge details, critical and unusual connection details, and proposals for fire and corrosion protection. When the comments of the other members of the design team have been received and assimilated, the scheme should be revised and the structural members redesigned as necessary.

3.2 Loads

Loads should be based on BS 648[3], BS 6399: Parts 1 and 3[4], and on CP3: Chapter V: Part 2[5] (or BS 6399 Part 2, in preparation).

Imposed loading should initially be taken as the highest statutory figures where options exist. The imposed load reductions allowed in the loading code should not be taken advantage of in the preliminary design except when assessing the load on foundations.

The load factors, γ_f, for use in design should be obtained from Table 1.

Temperature effects should also be considered where appropriate.

The effect of using beneficial load factors should be considered, and adverse load factors used if these will result in the use of a larger section.

Care should be taken not to underestimate the dead loads, and the following figures should be used to provide adequate loads in the absence of firm details:

floor finish (screed)	1.8kN/m^2 on plan
ceiling and service load	0.5kN/m^2 on plan
demountable lightweight partitions	1.0kN/m^2 on plan
blockwork partitions	2.5kN/m^2 on plan
external walling — curtain walling and glazing	0.5kN/m^2 on elevation
cavity walls (lightweight block/brick)	3.5kN/m^2 on elevation

Density of normal weight aggregate concrete should be taken as 24kN/m^3.
Density of lightweight aggregate concrete should be taken as 19kN/m^3.

3.3 Material selection

For multistorey construction in the UK, grade 50 steel may be used for beams acting compositely with the floors or where deflection does not govern the design; otherwise grade 43 steel should be used for beams. For columns, grade 50 steel should be considered where it is intended to reduce the sizes to a minimum. Grade 8.8 bolts should normally be used throughout.

3.4 Structural form and framing

The method for 'simple construction' as defined in BS 5950 should be used and the following measures adopted:

(a) provide braced construction by arranging suitable braced bays or cores deployed symmetrically wherever possible to provide stability against lateral forces in two directions approximately at right-angles to each other
(b) adopt a simple arrangement of slabs, beams and columns so that loads are carried to the foundations by the shortest and most direct routes using UC sections for the columns
(c) tie all columns effectively in two directions approximately at right-angles to each other at each floor and roof level. This may be achieved by the provision of beams or effective ties in continuous lines placed as close as practicable to the columns and to the edges of the floors and roofs
(d) select a floor construction that provides adequate lateral restraint to the beams (see subsection 3.8).
(e) allow for movement joints (see subsection 2.4)
(f) if large uninterrupted floor space is required and/or height is at a premium, choose a profiled-steel-decking composite floor construction that does not require propping. As a guide, limit the span of the floor to 2.5 – 3.6m; the span of the secondary beams to 8 – 12m; and the span of the primary beams to 5 – 7m
(g) in other cases, choose a precast or an *in situ* reinforced concrete floor, limiting their span as a guide to 5 – 6m, and the span of the beams to 6 – 8m.

The arrangement should take account of possible large openings for services and problems with foundations, e.g. columns immediately adjacent to site boundaries may require balanced or other special foundations.

3.5 Fire resistance

In the absence of specific information, choose a fire-resistance period of 1h for the superstructure and 2h for ground floor construction over a basement and the basement structure. This may be achieved by choosing one of the alternatives in Table 4.

Table 4 Fire protection

type of protection	period of fire resistance thickness in mm for 1h	2h
spray	20	35
boarding	15	30
intumescent paint (normally up to 1h)	1 – 5	—
reinforced concrete casing — loadbearing	50	50
reinforced concrete casing (1:2:4 mix) — non-loadbearing	25	25

More detailed guidance is given in:

Guidelines for the construction of fire resisting structural elements[9]
Fire protection for structural steel in building[10]
BS 5950: Part 8[6] (in preparation).

3.6 Corrosion protection

For multistorey buildings on non-polluted inland sites general guidance on systems for protection of steelwork in certain locations is given below. For other environments and for more detailed advice, reference should be made to BS 5493[11] and to publications from BSC, BCSA, ZDA and the Paintmakers Association. The general guidance is:

(a) *Steelwork integral with external cladding, particularly where not readily accessible for inspection and maintenance*
- (i) concrete encasement, or
- (ii) an applied coating system to give very long life such as:

  hot-dip galvanize to BS 729[12] (85μm) or
  blast clean SA2½, isocyanate pitch epoxy (450μm) (BS 5493 system reference SK8)

(b) *Internal steelwork not readily accessible, subject to condensation and/or significant corrosion risk*
A system to give long to very long life depending on corrosion risk such as:

blast clean SA2½, coal-tar epoxy (150μm), (SK5) or
blast clean SA2½, 2 pack zinc-rich epoxy (70μm), epoxy MI0 (125μm), (SL3)

(c) *External exposed steelwork, accessible*
A system to give medium life (or longer with appropriate maintenance cycles) such as :

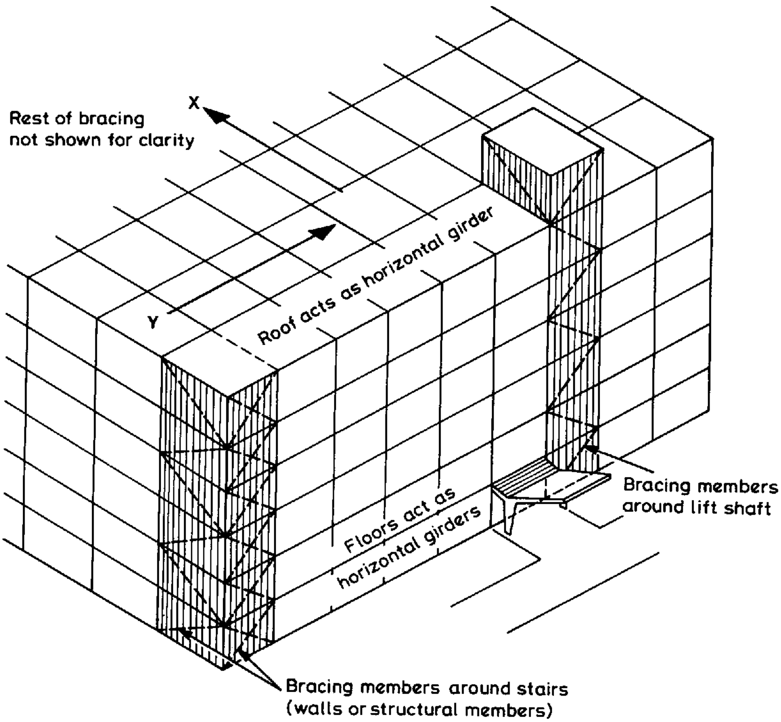
blast clean SA2½, HB zinc phosphate (70μm), modified alkyd (70μm), alkyd finish (35μm), (SF7)

(d) *Internal steelwork, heated building with negligible corrosion risk*
It is feasible to avoid treatment altogether in the right environment. Exposed steelwork not requiring fire protection will need a 'low life' coating system or better for decorative purposes. Otherwise, steelwork may require 'low life' protection to cover the period of delay before the cladding is erected. For sprayed fire protection systems the coating must be compatible.
Suitable systems include:
- (i) *shop applied*
  blast clean to SA2½, HB zinc phosphate (70μm)
- (ii) *site applied*
  manual clean C St 2, non-oxidizing 'grease' paint (100μm) or
  manual clean C St 2, HB pitch solution (150μm).

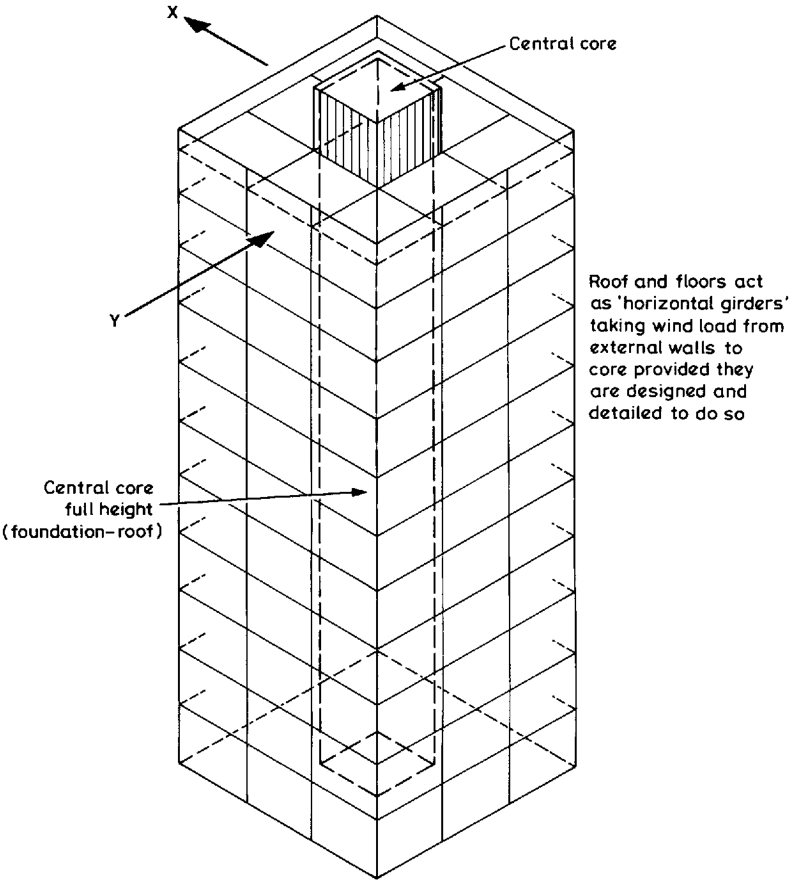
3.7 Bracing

Choose the location and form of bracing in accordance with the recommendations in clauses 2.2.3 and 3.4(*a*). Typical locations are shown on Figs. 1 and 2 for different shaped buildings.

The wind load or the notional horizontal forces on the structure, whichever are greater, should be assessed and divided into the number of bracing bays resisting the horizontal forces in each direction.



1 Braced frame rectangular or square on plan
Note that roof and floors will act as horizontal girders provided that they are designed and detailed to do so.



2 Braced frame square on plan—centre core

3.8 Flooring

It is essential at the start of the design of structural steelwork, to consider the details of the flooring system to be used, since these have a significant effect on the design of the structure.

Table 5 summarizes the salient features of the various types of flooring commonly used in the UK.

Table 5 Details of typical flooring systems and their relative merits

floor type	typical span range m	typical depth mm	construction time	degree of lateral restraint to beams	degree of diaphragm action	main areas of usage and remarks
timber	2.5 – 4	150 – 300	medium	poor	poor	domestic
in situ concrete	3 – 6	150 – 250	medium	very good	very good	all categories but not often used for multistorey steel construction, as formwork and propping are required
precast concrete	3 – 6	110 – 200	fast	fair – good	fair – good	all categories but cranage requirements and residual cambers should be considered
profiled steel decking composite with concrete topping	2.5 – 3.6 unpropped	110 – 150	fast	very good	very good	all categories especially multistorey commercial

Notes to Table 5

1. Timber floors should be designed to BS 5268[13].
2. *In situ* concrete floors should be designed to BS 8110[14] or to the IStructE/ICE *Manual for the design of reinforced concrete building structures*[15].
3. Precast concrete floors should be designed to BS 8110 and to the guides provided by the manufacturer of proprietory flooring systems.
4. Profiled-steel-decking/composite floors should be designed to BS 5950: Part 4[16] and to the literature provided by the manufacturers of the proprietory metal-decking systems.

4 Beams — bending only

4.1 Uncased non-composite beams

The first step in the design of these beams is to identify the restraint condition and the location of the loads applied to the beams in relation to the location of the restraints.

In this *Manual* the following four conditions are identified:

- Condition I: Full lateral restraint provided (e.g. beams supporting concrete floors)
  This condition will be satisfied if the frictional force or positive connection between the compression flange of the member and the floor it supports is capable of resisting a lateral force of at least 2½% of the force in the compression flange arising from the factored loads.
- Condition II: Full lateral restraint not provided, loads in any positions — conservative approach
  This may be used only for rolled universal sections. For other sections, or for a less conservative approach, beams should be designed using the procedures shown for conditions III or IV, as appropriate.
- Condition III: Full lateral restraint not provided and no load other than self-weight applied directly to the member between restraint points (e.g. primary beams restrained by secondary beams)
- Condition IV: Full lateral restraint not provided and load applied directly to the member between restraint points (e.g. primary edge beams restrained by secondary beams and supporting cladding loads)

The design procedures are described separately below for each condition.

4.2 Condition I: Full lateral restraint provided

Design procedure

(a) Calculate the factored load = 1.6 × imposed + 1.4 × dead, and then calculate the maximum factored bending moment (M_x), and the factored shear forces (F_v).

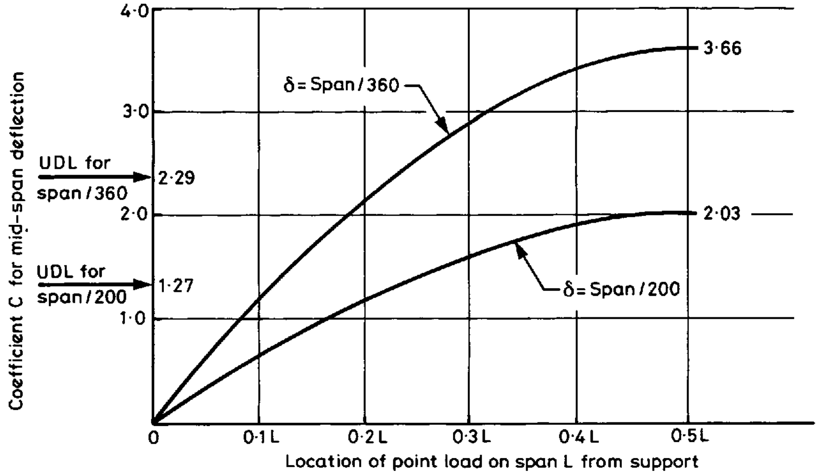
(b) Calculate the second moment of area (I) required to satisfy the deflection limitations described in clause 2.6.2. For simply supported beams:

$$I = C \times WL^2$$

where I is the second moment of area required in cm^4.
W is the total unfactored imposed distributed or point load in kN
L is the span in metres
and C is the deflection coefficient obtained for each loading from Fig. 3

When more than one load is imposed on the beam the principle of superposition may be used.

For cantilevers and continuous beams the deflections should be calculated from first principles taking into account the slopes at the supports and the ratio of the length of the cantilever to the span of its adjoining member.



3 *Deflection coefficient* C *for simply supported beams*

(c) Choose a section such that its second moment of area is greater than the required value and check that the moment capacity M_{cx} about its major axis $\geqslant M_x$

In order to choose a trial section that will not be critical in local buckling, it is necessary to note that elements and cross-sections have been classified as plastic, compact, semi-compact or slender in bending according to the limiting width/thickness ratios stated in Table 7 of BS 5950 and that different section modulii are used for calculating the moment capacities for different classes of sections.

In the blue book, each section has been classified for bending. It should be noted that the classification of a section may vary according to whether it is in bending and/or in compression, i.e. on the position of the neutral axis.

In order to assist the selection of suitable sections for use as beams in bending the classifications in Table 6 have been abstracted from the blue book.

Determine the value of the moment capacity M_{cx} about its major axis from:

$M_{cx} = p_y S_x$, but $\leqslant 1{\cdot}2\, p_y Z_x$ for plastic or compact sections, and

$M_{cx} = p_y Z_x$ for semi-compact or slender sections

where S_x is the plastic modulus of the section about the major axis,
Z_x is the elastic modulus of the section about the major axis, and
p_y is the design strength of the steel obtained from Table 2 according to the steel grade and flange thickness.

It should be noted that $p_y S_x$ will govern for UB sections, except as noted above.

Alternatively, M_{cx} may be obtained from the blue book where the second moments of area are also given.

Table 6 Section classification for bending only

All equal flanged rolled sections, and all CHS, SHS and RHS are plastic or compact for bending about the major axis (For RHS about minor axis see the blue book) *except* as follows:	
Grade 43 steel (semi-compact sections)	
356 × 368 × 129 UC	
305 × 305 × 97 UC	
152 × 152 × 23 UC	
Grade 50 steel (slender sections)	
250 × 250 × 6.3 SHS	
400 × 400 × 10 SHS	
Grade 50 steel (semi-compact sections)	
356 × 171 × 45 UB	152 × 76 × 17.88 RSC
254 × 146 × 31 UB	323.9 × 6.3 CHS
203 × 133 × 25 UB	355.6 × 8 CHS
356 × 368 × 153 UC	457.0 × 10.0 CHS
356 × 368 × 129 UC	508.0 × 10.0 CHS
305 × 305 × 97 UC	200 × 200 × 6.3 SHS
254 × 254 × 73 UC	250 × 250 × 8.0 SHS
203 × 203 × 46 UC	350 × 350 × 10.0 SHS
152 × 152 × 23 UC	400 × 400 × 12.5 SHS
	300 × 200 × 6.3 RHS

For slender sections a reduction in stress is necessary, and BS 5950 should be consulted.

In Appendix A tables are provided that give the resistance moments M_{cx} and the second moments of area I for a range of commonly used UB sections. A section may therefore be chosen from these tables which satisfies the two criteria for bending and deflection.

(d) Calculate the shear capacity P_v of the section chosen from $P_V = 0{\cdot}6\, p_y\, A_V$

where p_y is obtained from Table 2, and

A_V is the shear area defined as follows:

for load parallel to web for I,H, channel and RHS	$= tD$
for solid bars and plates	$= 0.9A$
for circular hollow section	$= 0.6A$
for other sections	$= 0.9A_o$

where t is the web thickness (note: use both webs for RHS)

D is the overall depth of section

A is the area of the section, and

A_o is the area of the rectilinear elements of the section that have their longest dimension in the direction parallel to the load.

Alternatively, P_V may be obtained from the blue book.

No further checks are required if the shear force $F_V < 0.6\ P_V$

Where $F_V > 0.6\ P_V$ the moment capacity should be reduced. This will be significant only if high shear and high moment occur together at the same location on the beam, in which case the section size should be increased.

Alternatively, for all symmetrical sections the following simplified formula may be used:

$$\text{The reduced value of } M_{cx} = M_{cx} - \left(\frac{2 \cdot 5F_v}{P_v} - 1 \cdot 5\right) p_y \frac{D^2 t}{4}$$

(e) Check for web bearing and buckling.

If web cleats or end plates are used for the end connections of the beams then no check is required. For other types of connections, checks should be carried out in accordance with the provisions of BS 5950 or the tables in the blue book should be used.

4.3 Condition II: Full lateral restraint not provided, loads in any position: conservative method

All beams designed by this method should also satisfy the requirements of Condition I for bending, deflection, shear, web bearing and buckling.

Design procedure

(a) Calculate the factored load = 1.6 × imposed + 1.4 × dead, and then calculate the maximum factored bending moments (M_x) and the factored shear forces (F_v).

(b) Calculate the second moment of area (I) required to satisfy the deflection limitations described in clause 2.6.2. For simply supported beams, use the method described in clause 4.2 (*b*).

(c) Determine the effective length L_E from the two cases:

- *Beams with lateral restraints at their ends only*

The effective length L_E should be obtained from Table 7 according to the conditions of restraints at their ends. If the conditions of restraint differ at each end then a mean value of L_E may be taken.

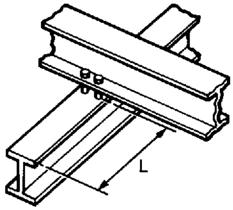
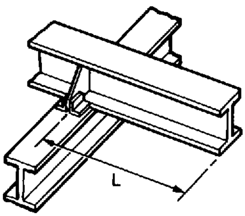
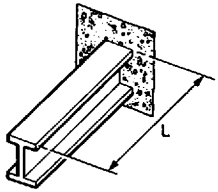
For cantilevers the effective length L_E should be obtained from Table 8.

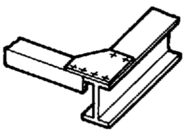
Table 7 Effective length of beams L_E

conditions of restraint at the ends of the beams		loading conditions: normal	loading conditions: destabilizing (see note 3)
compression flange laterally restrained; beam fully restrained against torsion	both flanges fully restrained against rotation on plan	$0{\cdot}7L$	$0{\cdot}85L$
	both flanges partially restrained against rotation on plan	$0{\cdot}85L$	$1{\cdot}0L$
	both flanges free to rotate on plan	$1{\cdot}0L$	$1{\cdot}2L$
compression flange laterally unrestrained; both flanges free to rotate on plan	restraint against torsion provided only by positive connection of bottom flange to supports	$1{\cdot}0L+2D$	$1{\cdot}2L+2D$
	restraint against torsion provided only by dead bearing of bottom flange on supports	$1{\cdot}2L+2D$	$1{\cdot}4L+2D$

Notes to Table 7
1. D is the depth of the beam.
2. L is the length of the beam between its ends.
3. It should be noted that destabilizing load conditions exist when a load is applied to the compression flange of a beam or the tension flange of a cantilever and both the load and the flange are free to deflect laterally (and possibly rotationally also) relative to the centroid of the beam.

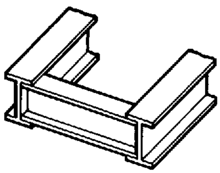
Table 8 Effective length of cantilever L_E

Restraint conditions		Loading conditions	
At support	**At tip**	**Normal**	**Destabilizing**
Continuous with lateral restraint only  	Free	3·0 L	7·5 L
	Laterally restrained on top flange only	2·7 L	7·5 L
	Torsionally restrained only	2·4 L	4·5 L
	Laterally and torsionally restrained	2·1 L	3·6 L
Continuous with lateral and torsional restraint  	Free	1·0 L	2·5 L
	Laterally restrained on top flange only	0·9 L	2·5 L
	Torsionally restrained only	0·8 L	1·5 L
	Laterally and torsionally restrained	0·7 L	1·2 L
Built-in laterally and torsionally  	Free	0·8 L	1·4 L
	Laterally restrained on top flange only	0·7 L	1·4 L
	Torsionally restrained only	0·6 L	0·6 L
	Laterally and torsionally restrained	0·5 L	0·5 L



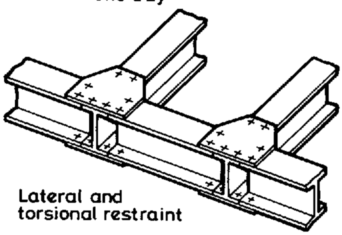
Top flange restraint

Face beams extending over several bays



Torsional restraint

Braced laterally in at least one bay



Lateral and torsional restraint

Note : When values from this table are used for L_E the equivalent uniform moment factor, m , and the slenderness correction factor, n , should be taken as 1·0

- ***Beams with effective intermediate lateral restraints as well as at their ends***
  Provided that the lateral restraints have been designed to be adequate then the effective lengths L_E of the parts of the beam may be obtained from the following:

  (i) *Part of beam between restraints*
  The effective length L_E of this part of the beam should be taken as the actual distance between the restraints.

  (ii) *Part of the beam between the end of the beam and the first internal lateral restraint*
  The effective length L_E should be taken as the mean of the value given by (i) and the value given by Table 7 for the conditions of restraint at the support, taking L as the distance between the restraint and the support in both cases.

It is most important to design the lateral restraints so that they have adequate stiffness and strength. Restraints may be deemed to provide adequate strength if they are capable of resisting a lateral force of not less than 2½% of the maximum factored force in the compression flange or chord. Where several members share a common restraint, the minimum total lateral force may be taken as the sum of those derived from the largest three members.

When a series of two or more parallel beams require a lateral restraint at intervals, it is not adequate merely to tie the compression flanges together such that the members become mutually dependent. Adequate restraint to any beam will be achieved only if the beam supports and the restraining members are held by a robust part of the structure or held in a fixed relationship to each other by means of triangulated bracing.

(d) Choose a trial section and grade of steel and check that the maximum M_x on any portion of the beam between adjacent lateral restraints does not exceed the buckling resistance moment M_b of the section obtained from:

$$M_b = p_b S_x$$

where p_b is the bending strength of the member and
S_x is the plastic modulus of the section about the x – x axis.

The bending strength p_b of the trial section is obtained from the tables in Appendix B for the design strength p_y, the slenderness λ and the torsional index x

where p_y is the design strength obtained from Table 2 according to the grade of steel and thickness of the flange of the chosen section.

$$\lambda = \left(\frac{L_E}{r}\right) n$$

where L_E is the effective length obtained in (*c*)
r is the radius of gyration of the section about its minor axis, and
n for beams without intermediate lateral restraints may be taken as:

0.86 for central point loads
0.94 for all other loads

For beams with intermediate lateral restraints, cantilevers and beams subject to destabilizing loads n should be taken as 1.0.
Less conservative values of n may be obtained from Tables 12 or 13.

x is the torsional index which may be taken as the D/T ratio where D is the depth of the section and T is the thickness of the flange as obtained from the blue book.

The buckling resistance moment M_b should be calculated for each portion of beam from $M_b = p_b S_x$. If this is less than the corresponding maximum M_x on that portion of beam a larger section or higher grade of steel should be chosen or additional restraints provided and the calculation repeated.

In Appendix A tables are provided that give the buckling resistance moments for commonly used UBs for a range of effective lengths L_E. The tables also show the critical values of L_E for each UB at which $p_b = p_y$.

(e) Check that the beam complies with the requirements for bending and deflection using the procedure detailed in clauses 4.2 *(b)* and *(c)*

(f) Check that the shear capacity P_v of the sections exceeds the factored shear forces (F_v) using the procedure detailed in clause 4.2(*d*).

(g) Check for web bearing and buckling as detailed in clause 4.2(*e*).

4.4 Condition III: Full lateral restraint not provided and no load other than self-weight applied directly to the member between restraint points (e.g. primary beams restrained by secondary beams)

All beams designed by this method should also satisfy the requirements of Condition I for bending, deflection, shear, web bearing and buckling.

Design procedure

(a) Calculate the factored load = 1.6 × imposed + 1.4 × dead and then calculate the maximum factored bending moments (M_x) and the factored shear forces (F_v).

(b) Calculate the second moment of area (I) required to satisfy the deflection limitations described in clause 2.6.2. For simply supported beams, use the method described in clause 4.2 (*b*).

(c) Determine the effective length L_E as described in clause 4.3 (*c*).

(d) Choose a trial section and grade of steel and check that the equivalent uniform factored moment $\bar{M}$ on any portion of beam between adjacent lateral restraints, does not exceed the buckling resistance moment M_b of the section chosen. $\bar{M}$ is obtained from $\bar{M} = mM_A$

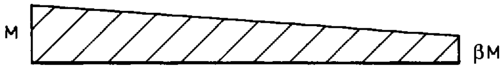
where m is the equivalent uniform moment factor obtained from Table 9, and
M_A is the maximum M_x on the portion of the member being considered.

The buckling resistance moment M_b of the section is obtained from $M_b = p_b S_x$

where p_b is the bending strength of the member, and
S_x is the plastic modulus of the section about the x-x axis

The bending strength p_b is obtained from Table 11 for the design strength p_y and the equivalent slenderness λ_{LT}
p_y is the design strength obtained from Table 2 according to the grade of steel and thickness of the flange of the chosen section, and

Table 9 Equivalent uniform moment factor, *m*

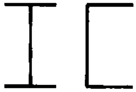
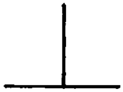
Bending moment diagram between restraints	β	m
β positive (single-curvature bending) 	1·0	1·0
	0·9	0·95
	0·8	0·90
	0·7	0·85
	0·6	0·80
	0·5	0·76
	0·4	0·72
	0·3	0·68
	0·2	0·64
	0·1	0·60
β negative (double-curvature bending) 	0·0	0·57
	−0·1	0·54
	−0·2	0·51
	−0·3	0·48
	−0·4	0·45
	−0·5 to −1·0	0·43

Notes to Table 9
1. β is the ratio of the smaller end moment to the larger end moment.
2. For cantilevers and members subject to destabilizing loads m = 1·0.
3. For sections other than those with equal uniform flanges m = 1·0.

the equivalent slenderness $\lambda_{LT} = nuv\lambda$
where λ is the effective length L_E obtained as described in clause 4.3 (*c*) divided by the radius of the gyration r_y of the chosen section about its minor axis
n is the slenderness correction factor which is equal to 1.0 for Condition III
u is the buckling parameter which may be taken as 0.9 for all rolled I-, H- or channel sections, 1.0 for all other sections, or may be obtained from the section property tables in the blue book
v is a slenderness factor which may be obtained from Table 10 for all symetric flanged members uniform, and tees or Table 14 of BS 5950 for all other sections.

To obtain v from Table 10, N may be taken as 0.5 for all symmetrically flanged sections (i.e. universal beams, columns or channels), and 1.0 and 0.0 as appropriate for T-sections, λ/x is obtained from λ determined as above and x as for Condition II (see clause 4.3(*d*))

Table 10 Slenderness factor, v, for flanged beams of uniform section

	Compression 		Compression 
N / λ/x	1·0	0·5	0·0
0·5	0·79	1·00	12·67
1·0	0·78	0·99	6·36
1·5	0·77	0·97	4·27
2·0	0·76	0·96	3·24
2·5	0·75	0·93	2·62
3·0	0·74	0·91	2·21
3·5	0·72	0·89	1·93
4·0	0·71	0·86	1·71
4·5	0·69	0·84	1·55
5·0	0·68	0·82	1·41
5·5	0·66	0·79	1·31
6·0	0·65	0·77	1·22
6·5	0·64	0·75	1·14
7·0	0·63	0·73	1·08
7·5	0·61	0·72	1·02
8·0	0·60	0·70	0·98
8·5	0·59	0·68	0·93
9·0	0·58	0·67	0·90
9·5	0·57	0·65	0·86
10·0	0·56	0·64	0·83
11·0	0·54	0·61	0·78
12·0	0·53	0·59	0·73
13·0	0·51	0·57	0·69
14·0	0·50	0·55	0·66
15·0	0·49	0·53	0·63
16·0	0·47	0·52	0·61
17·0	0·46	0·50	0·58
18·0	0·45	0·49	0·56
19·0	0·44	0·48	0·55
20·0	0·43	0·47	0·53

Notes to Table 10

1. $N = \dfrac{I_{cf}}{I_{cf} + I_{tf}}$

where I_{cf} and I_{tf}, are the second moments of area of the compression and tension flanges, respectively, about the minor axis of the section.

2. For intermediate values to the right of the heavy line in the Table, v should be determined from the general formulae given in B.2.5 of BS 5950.

3. Interpolation horizontally across the table is not permissible.

The buckling resistance moment M_b should be calculated for each portion of beam from $M_b = p_b S_x$.

If this is less than the corresponding equivalent uniform factored moment $\overline{M}$ on that portion of beam, a larger section or higher grade of steel should be chosen or additional restraints provided and the calculation repeated.

(e) Check that the beam complies with the requirements for bending and deflection using the procedure detailed in clauses 4.2 (*b*) and (*c*).

(f) Check that the shear capacity P_v of the sections exceeds the factored shear forces (F_v) using the procedure detailed in clause 4.2 (*d*).

(g) Check for web bearing and buckling as detailed in clause 4.2 (*e*).

4.5 Condition IV: Full lateral restraint not provided and load applied directly to the member between restraint points (e.g. primary edge beams restrained by secondary and beams supporting cladding loads)

All beams designed by this method should also satisfy the requirements of Condition I for bending deflection, shear, web bearing and buckling.

Design procedure

(a) Calculate the factored load = 1.6 × imposed + 1.4 × dead, and then calculate the maximum factored bending moments (M_x) and the factored shear forces (F_v).

(b) Calculate the second moment of area (I) required to satisfy the deflection limitations described in clause 2.6.2. For simply supported beams, use the method described in clause 4.2 (*b*).

(c) Determine the effective length L_E as described in clause 4.3 (*c*).

(d) Choose a trial section and grade of steel and check that the equivalent uniform factored moment $\overline{M}$ on any portion of beam between adjacent lateral restraints does not exceed the buckling resistance M_b of the section chosen. $\overline{M}$ is obtained from $\overline{M} = mM_A$ where m is the equivalent uniform moment factor, which is equal to 1.0 for Condition IV, and M_A is the maximum M_x on the portion of the member being considered.

The buckling resistance moment M_b of the section is obtained from

$$M_b = p_b S_x$$

where p_b is the bending strength of the member and

S_x is the plastic modulus of the sections about the x-x axis.

The bending strength p_b of the trial section is obtained from Table 11 for the design strength p_y and of the equivalent slenderness λ_{LT}.

Table 11a gives the limiting values of slendernesses λ_{LO} at which $p_b = p_y$. The design strength p_y is obtained from Table 2 according to the grade of steel and thickness of the flange of the chosen section, and the equivalent slenderness $\lambda_{LT} = nuv\lambda$

where λ is the effective length L_E obtained as described in clause 4.3 (*c*) divided by the radius of the gyration r_y of the chosen section about its minor axis.

n is the slenderness factor obtained from Tables 12, 13 and 14. For cantilevers and destablilizing loads $n = 1{\cdot}0$

u is the buckling parameter which may be taken as 0.9 for all rolled I-, H- or channel sections, 1.0 for all other sections or may be obtained from the section property tables in the blue book.
v is a slenderness factor, which may be obtained from Table 10 for all flanged members of uniform section, or Table 14 of BS 5950 for all other sections.

To obtain v from Table 10, N may be taken as 0.5 for all flanged members uniform about one axis (i.e. universal beams, columns or channels), and λ/x is obtained from λ determined as above and x is obtained from the blue book.
The buckling resistance moment M_b should be calculated for each portion of beam from $M_b = p_b S_x$. If this is less than the corresponding equivalent uniform factored moment $\overline{M}$ on that portion of beam, a larger section or higher grade of steel should be chosen or additional restraints provided and the calculation repeated.

(e) Check that the beam complies with the requirements for bending and deflection using the procedure detailed in clauses 4.2 (*b*) and (*c*).

(f) Check that the shear capacity P_v of the sections exceeds the factored shear forces (F_v) using the procedure detailed in clause 4.2(*d*).

(g) Check for web bearing ~~and~~ buckling as detailed in clause 4.2(*e*).

Table 11 Bending strength, p_b, for rolled sections in N/mm²

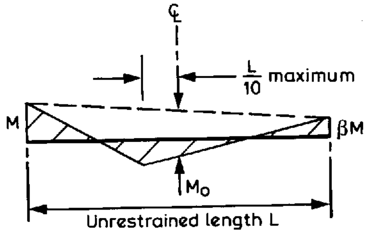
λ_{LT} \ p_y	245	255	265	275	325	340	345	355	400	415	430	450
30	245	255	265	275	325	340	345	355	395	408	421	438
35	245	255	265	273	316	328	332	341	378	390	402	418
40	238	246	254	262	302	313	317	325	359	371	382	397
45	227	235	242	250	287	298	302	309	340	350	361	374
50	217	224	231	238	272	282	285	292	320	329	338	350
55	206	213	219	226	257	266	268	274	299	307	315	326
60	195	201	207	213	241	249	251	257	278	285	292	300
65	185	190	196	201	225	232	234	239	257	263	269	276
70	174	179	184	188	210	216	218	222	237	242	247	253
75	164	168	172	176	195	200	202	205	219	223	226	231
80	154	158	161	165	181	186	187	190	201	204	208	212
85	144	147	151	154	168	172	173	175	185	188	190	194
90	135	138	141	144	156	159	160	162	170	173	175	178
95	126	129	131	134	144	147	148	150	157	159	161	163
100	118	121	123	125	134	137	137	139	145	147	148	150
105	111	113	115	117	125	127	128	129	134	136	137	139
110	104	106	107	109	116	118	119	120	124	126	127	128
115	97	99	101	102	108	110	110	111	115	117	118	119
120	91	93	94	96	101	103	103	104	107	108	109	111
125	86	87	89	90	95	96	96	97	100	101	102	103
130	81	82	83	84	89	90	90	91	94	94	95	96
135	76	77	78	79	83	84	85	85	88	88	89	90
140	72	73	74	75	78	79	80	80	82	83	84	84
145	68	69	70	71	74	75	75	75	77	78	79	79
150	64	65	66	67	70	70	71	71	73	73	74	75
155	61	62	62	63	66	66	67	67	69	69	70	70
160	58	59	59	60	62	63	63	63	65	65	66	66
165	55	56	56	57	59	60	60	60	61	62	62	63
170	52	53	53	54	56	56	57	57	58	59	59	59
175	50	50	51	51	53	54	54	55	56	56	56	56
180	47	48	48	49	51	51	51	51	52	53	53	53
185	45	46	46	46	48	49	49	49	50	50	50	51
190	43	44	44	44	46	46	46	47	48	48	48	48
195	41	42	42	42	44	44	44	44	45	46	46	46
200	39	40	40	40	42	42	42	42	43	43	44	44
210	36	37	37	37	38	39	39	39	39	40	40	40
220	33	34	34	34	35	35	35	36	36	36	37	37
230	31	31	31	31	32	33	33	33	33	33	34	34
240	29	29	29	29	30	30	30	30	31	31	31	31
250	27	27	27	27	28	28	28	28	29	29	29	29

Note: p_b may be taken as p_y provided that λ_{LT} does not exceed λ_{LO} as shown in Table 11a.

Table 11a Limiting equivalent slendernesses λ_{LO}

p_y	245	265	275	325	340	355	415	430	450
λ_{LO}	36·3	35·0	34·3	31·6	30·9	30·2	27·9	27·4	26·8

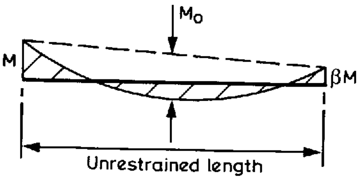
Table 12 Slenderness correction factor, n, for members with applied loading concentrated within the middle fifth of the unrestrained length



$\gamma = M/M_o$	β positive						β negative				
	1·0	0·8	0·6	0·4	0·2	0·0	−0·2	−0·4	−0·6	−0·8	−1·0
+50.00	1·00	0·96	0·92	0·87	0·82	0·77	0·72	0·67	0·66	0·66	0·65
+10·00	0·99	0·99	0·94	0·90	0·85	0·80	0·75	0·69	0·68	0·68	0·67
+5·00	0·98	0·98	0·97	0·93	0·89	0·84	0·79	0·73	0·71	0·70	0·70
+2·00	0·96	0·95	0·95	0·95	0·94	0·94	0·89	0·84	0·79	0·77	0·76
+1·50	0·95	0·95	0·94	0·94	0·93	0·93	0·92	0·90	0·85	0·80	0·80
+1·00	0·93	0·92	0·92	0·92	0·92	0·91	0·91	0·91	0·91	0·92	0·92
+0·50	0·90	0·90	0·90	0·89	0·89	0·89	0·89	0·89	0·88	0·88	0·88
0·00	0·86	0·86	0·86	0·86	0·86	0·86	0·86	0·86	0·86	0·86	0·86
−0·10	0·85	0·85	0·85	0·85	0·85	0·86	0·86	0·86	0·86	0·86	0·86
−0·20	0·83	0·83	0·83	0·84	0·84	0·85	0·85	0·85	0·86	0·86	0·86
−0·30	0·81	0·82	0·82	0·83	0·83	0·84	0·85	0·85	0·86	0·86	0·87
0·40	0·79	0·80	0·81	0·81	0·82	0·83	0·84	0·85	0·85	0·86	0·87
−0·50	0·77	0·78	0·79	0·80	0·82	0·83	0·85	0·86	0·86	0·87	0·88
−0·60	0·62	0·66	0·72	0·77	0·80	0·82	0·84	0·85	0·86	0·87	0·88
−0·70	0·56	0·56	0·61	0·67	0·73	0·79	0·83	0·85	0·87	0·88	0·89
−0·80	0·56	0·53	0·54	0·59	0·65	0·71	0·77	0·83	0·89	0·90	0·90
−0·90	0·59	0·57	0·54	0·53	0·57	0·64	0·71	0·77	0·84	0·88	0·91
−1·00	0·62	0·58	0·54	0·52	0·54	0·59	0·66	0·72	0·80	0·85	0·92
−1·10	0·66	0·62	0·57	0·54	0·54	0·57	0·63	0·68	0·76	0·83	0·89
−1·20	0·70	0·66	0·60	0·55	0·54	0·55	0·60	0·65	0·73	0·80	0·87
−1·30	0·73	0·69	0·63	0·57	0·55	0·54	0·57	0·61	0·69	0·77	0·83
−1·40	0·74	0·70	0·64	0·58	0·56	0·54	0·55	0·60	0·66	0·74	0·81
−1·50	0·75	0·70	0·64	0·59	0·56	0·54	0·55	0·59	0·65	0·73	0·80
−1·60	0·76	0·72	0·65	0·60	0·57	0·55	0·55	0·58	0·64	0·72	0·80
−1·70	0·77	0·74	0·66	0·61	0·58	0·56	0·55	0·58	0·63	0·70	0·78
−1·80	0·79	0·77	0·68	0·63	0·59	0·56	0·56	0·57	0·62	0·69	0·76
−1·90	0·80	0·79	0·69	0·64	0·60	0·57	0·56	0·57	0·61	0·67	0·75
−2·00	0·81	0·81	0·70	0·65	0·61	0·58	0·56	0·56	0·60	0·66	0·74
−5·00	0·93	0·89	0·83	0·77	0·72	0·67	0·64	0·61	0·60	0·62	0·65
−50·00	0·99	0·95	0·90	0·86	0·79	0·74	0·70	0·67	0·64	0·63	0·65
infinity	1·00	0·96	0·91	0·86	0·82	0·77	0·72	0·68	0·65	0·65	0·65

Notes to Table 12
1. All hogging moments are +ve.
2. β is defined in Table 9.
3. M_o is the midlength moment on a simply supported span equal to the unrestrained length (see Table 14).
4. The values of n in this table apply only to members of uniform section.
5. Values for intermediate values of β and γ may be interpolated.
6. When n from this table is used, m = 1.00.

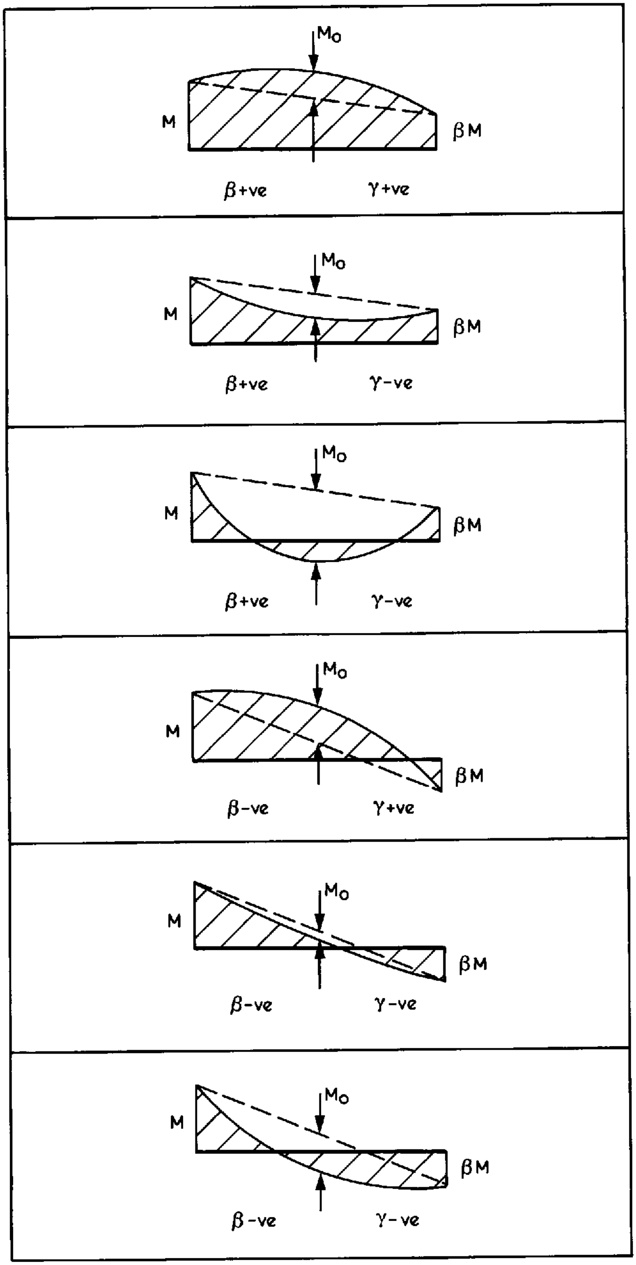
Table 13 Slenderness correction factor, n, for members with applied loading other than as for Table 12



$\gamma = M/M_o$	β positive						β negative				
	1·0	0·8	0·6	0·4	0·2	0·0	−0·2	−0·4	−0·6	−0·8	−1·0
+50.00	1·00	0·96	0·92	0·87	0·83	0·77	0·72	0·67	0·66	0·66	0·65
+10·00	0·99	0·98	0·95	0·91	0·86	0·81	0·76	0·70	0·68	0·68	0·67
+5·00	0·99	0·98	0·97	0·94	0·90	0·85	0·80	0·75	0·71	0·70	0·70
+2·00	0·98	0·98	0·97	0·96	0·94	0·92	0·90	0·86	0·82	0·78	0·76
+1·50	0·97	0·97	0·97	0·96	0·95	0·93	0·92	0·89	0·86	0·83	0·79
+1·00	0·97	0·97	0·97	0·96	0·96	0·95	0·94	0·93	0·93	0·91	0·89
+0·50	0·96	0·96	0·96	0·96	0·96	0·95	0·94	0·94	0·94	0·93	0·92
0·00	0·94	0·94	0·94	0·94	0·94	0·94	0·94	0·94	0·94	0·94	0·94
−0·10	0·93	0·93	0·93	0·93	0·94	0·94	0·94	0·94	0·94	0·94	0·94
−0·20	0·92	0·92	0·92	0·92	0·93	0·93	0·93	0·93	0·94	0·94	0·93
−0·30	0·91	0·91	0·92	0·92	0·93	0·93	0·93	0·93	0·94	0·94	0·94
0·40	0·90	0·90	0·91	0·91	0·92	0·92	0·92	0·92	0·93	0·93	0·93
−0·50	0·89	0·90	0·91	0·91	0·92	0·92	0·92	0·92	0·92	0·92	0·92
−0·60	0·71	0·77	0·84	0·87	0·89	0·91	0·92	0·92	0·92	0·92	0·92
−0·70	0·57	0·64	0·70	0·77	0·82	0·87	0·89	0·91	0·92	0·92	0·91
−0·80	0·47	0·52	0·59	0·67	0·73	0·80	0·86	0·90	0·92	0·92	0·92
−0·90	0·47	0·46	0·50	0·58	0·65	0·73	0·80	0·87	0·90	0·90	0·90
−1·00	0·50	0·48	0·46	0·51	0·58	0·66	0·73	0·81	0·87	0·89	0·89
−1·10	0·54	0·51	0·48	0·49	0·54	0·61	0·69	0·77	0·83	0·87	0·88
−1·20	0·57	0·54	0·50	0·47	0·51	0·56	0·64	0·73	0·80	0·84	0·87
−1·30	0·61	0·56	0·52	0·47	0·49	0·53	0·61	0·70	0·77	0·82	0·86
−1·40	0·64	0·59	0·55	0·49	0·48	0·51	0·58	0·67	0·74	0·79	0·85
−1·50	0·67	0·62	0·57	0·51	0·47	0·49	0·56	0·64	0·71	0·77	0·84
−1·60	0·69	0·64	0·59	0·52	0·48	0·50	0·55	0·63	0·69	0·76	0·83
−1·70	0·71	0·66	0·60	0·54	0·50	0·51	0·55	0·61	0·68	0·74	0·82
−1·80	0·74	0·69	0·62	0·55	0·51	0·51	0·54	0·60	0·66	0·73	0·81
−1·90	0·76	0·71	0·63	0·57	0·53	0·52	0·54	0·58	0·65	0·71	0·80
−2·00	0·78	0·73	0·65	0·58	0·54	0·53	0·53	0·57	0·63	0·70	0·79
−5·00	0·91	0·86	0·80	0·74	0·70	0·65	0·62	0·59	0·58	0·61	0·67
−50·00	0·99	0·95	0·89	0·84	0·79	0·74	0·70	0·66	0·63	0·62	0·65
infinity	1·00	0·96	0·91	0·86	0·82	0·77	0·72	0·68	0·65	0·65	0·65

Notes to Table 13
1. All hogging moments are +ve.
2. β is defined in Table 9.
3. M_o is the midlength moment on a simply supported span equal to the unrestrained length (see Table 14).
4. The values of n in this table apply only to members of uniform section.
5. Values for intermediate values of β and γ may be interpolated.
6. When n from this table is used, m = 1.00.

Table 14 Typical moment diagrams between adjacent points of lateral restraint



4.6 Cased beams

4.6.1 Introduction
This subsection describes the design of cased beams that are subject to bending only and which satisfy the conditions in clause 4.6.2. The design of cased beams not satisfying these conditions should be carried out by reference to BS 5950. To allow for the additional stiffening afforded by the concrete casing the design should be carried out by following the design procedure described in clause 4.6.3.

4.6.2 Conditions
The conditions to be satisfied to permit the stiffening effect of concrete casing to be taken into account are as follows:

(a) The steel section is either:

(i) a single rolled section or a fabricated section with equal I- or H-flanges, or
(ii) rolled equal channel sections arranged back to back, with a maximum separation not exceeding half the depth of the section.

(b) The dimensions of the steel sections do not exceed a depth of 1000mm (parallel to the web(s)) or a width of 500mm.
(c) The steel section is unpainted and is free from oil, grease, dirt and loose rust and millscale.
(d) There is a minimum rectangle of concrete casing consisting of well compacted ordinary dense concrete of at least grade 25 to BS 8110 and extends the full length of the steel member and its connections.
(e) The concrete casing may be chamfered at corners but should provide cover to the outer faces and edges of at least 50mm.
(f) The casing is reinforced with either:

(i) D98 fabric complying with BS 4483[17], or
(ii) a cage of closed links and longitudinal bars using steel reinforcement or wire not less than 5mm diameter and complying with BS 4449[18] or BS 4482[19], at a maximum spacing of 200mm.

The reinforcement should pass through the centre of the concrete cover of the flanges and should be detailed to comply with BS 8110.

(g) The effective length L_E of the cased section is limited to the least of $40b_c$, or $(100b_c^2/d_c)$ or $250\,r$.
where b_c and d_c are, respectively, the minimum width and depth of solid casing, and
r is the minimum radius of gyration of the uncased steel section.

4.6.3 Design procedure
The cased beams should be designed using the procedures for uncased beams taking into account the following additional provisions:

(a) The second moments of area I_{cs} for the cased section for calculations of deflection may be taken as:

$$I_s + \frac{I_c - I_s}{\alpha_e}$$

where I_s is the second moment of area of steel section
I_c is the second moment of area of gross concrete section
α_e is the ratio of modulus of elasticity of steel and concrete, which may generally be taken as having a value of 15.

(b) In the calculations of slenderness, the radius of gyration of the cased section should be taken as the greater of:

(i) $0{\cdot}2\ (B + 100)$ mm, or
(ii) r_y of the uncased section

where B is the width of the steel flanges.

(c) The buckling resistance moment M_b of the cased section should be limited to 1.5 times that permitted for the uncased section.

4.7 Single angles

Design procedure

Angles that are subject to bending only and free to buckle about their weakest axis may be designed using the procedures given for uncased beams provided that the buckling resistance moment M_b is taken as:

$$M_b = 0.8\ p_y Z \text{ for } L/r_{vv} < 100$$
$$M_b = 0.7\ p_y Z \text{ for } L/r_{vv} < 180$$
$$M_b = 0.6\ p_y Z \text{ for } L/r_{vv} < 300$$

where Z is the elastic modulus about the appropriate axis
r_{vv} is the radius of gyration about the weakest axis, and
L is the unrestrained length.

Linear interpolation may be used to obtain intermediate values.

4.8 Hollow sections

Design procedure

The procedure given in subsection 4.2 for condition I (full lateral restraint provided) may be followed provided that λ (i.e. L_E/r_y) is within the limits shown below.

D/B	λ	D/B	λ
1	∞	3	$(225 \times 275)/p_y$
2	$(350 \times 275)/p_y$	4	$(170 \times 275)/p_y$

where D and B are overall depth and breadth of box section, respectively.

For a circular hollow section $D/B = 1$.

4.9 Composite beams

The design of composite beams is a lengthy iterative process and is thus ideally suited to computer analysis. For grade 50 steel and slab depths in the range of 110 – 140 mm, approximate span/overall depths of construction ratios of $L/19 - L/23$ may be used for UB sections and $L/22 - L/29$ for UC sections, where L is the span of the beam.

Section 1 of *Steel framed multistorey buildings: design recommendations for composite floors and beams using steel decks*[20] contains tables that may be used for the initial selection of the beam sizes.

5 Braced multistorey buildings — columns in compression and bending

5.1 Uncased columns

This Section describes the design of uncased columns for braced multistorey construction which are subject to compression and bending.

Two cases are considered:

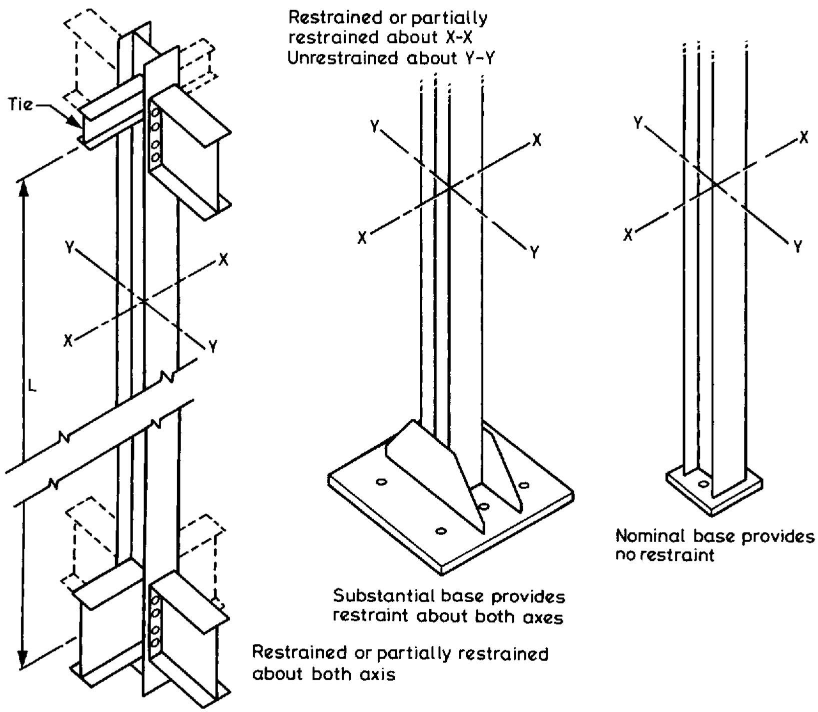
- Case I: columns braced in both directions and subject only to nominal moments applicable to simple construction
- Case II: columns braced in both directions and subject to applied moments other than nominal moments.

For both of these cases an iterative process is used requiring selection and subsequent checking of a trial section.

The first step is to determine the effective lengths L_E of the column about its major and minor axis.

5.2 Determination of effective length of columns

For braced multistorey buildings the columns are held in position, so that the effective length L_E to be used in design depends on the degree of restraint in direction (i.e. rotational restraint) afforded by the beams attached to the columns at each floor level or the foundations. Fig. 4 illustrates typical joint and foundation restraint conditions.



Note: If the depth of the plate at the end of the beam is less than 0.6 × the depth of the beam, then no directional restraint is provided.

To determine the degree of restraint about each axis at each end of the column the joint restraint coefficient, K, about each axis may be assessed from:

$$k = \frac{\text{total stiffness of column members at joint}}{\text{total stiffness of all members at joint}}$$

where member stiffness = I/L

Common practice suggests that:

- if $k < 0{\cdot}5$, column is restrained in direction
- if $0.5 < k < 0.8$, column is partially restrained in direction
- if $k > 0.8$, column is unrestrained in direction
- nominal foundation – column is unrestrained in direction
- substantial foundation – column is restrained in direction.

From the degree of restraint assessed at each end, the effective length L_E should be determined from in Table 15, where L should be taken as the distance between the points of effective restraints on each axis.

Table 15 Effective length L_E — braced frame

condition of restraint effective length (in plane under consideration)	L_E
both ends unrestrained in direction, or one end partially restrained in direction and the other end unrestrained in direction.	$1.0L$
both ends partially restrained in direction, or one end restrained in direction and the other partially restrained in direction	$0.85L$
both ends restrained in direction	$0.70L$

5.3 Column selection

Before selecting a trial section it is necessary to note that elements and cross-sections have been classified as plastic, compact, semi-compact or slender in combined compression and bending according to the limiting width/thickness ratios stated in Table 7 of BS 5950. In this *Manual* slender sections are not considered for use in Case I.

Slender sections have been identified (for axial compression only) in the blue book. In order to assist the selection of suitable sections as columns for simple multistorey construction it should be noted that all UCs, RSCs, and CHSs and most RHSs, together with the universal beam sections shown in Table 16, which are not slender, could be chosen.

Table 16 Non-slender UB sections in compression

grade 43	grade 50
914 × 419 × 388	
610 × 305 × 238 and × 179	610 × 305 × 238 and × 179
533 × 210 × 122	356 × 171 × 67
457 × 191 × 98 and × 89	305 × 127 × 48 and × 42
457 × 152 × 82	254 × 146 × 43 and × 37
406 × 178 × 74	203 × 133 × 30 and × 25
356 × 171 × 67	
305 × 165 × 54	
305 × 102 × 28 and × 25	
305 × 127 × 48 and × 42 × 37	
254 × 146 × 43 and × 37 × 31	
254 × 102 × 28 and × 25 × 22	
203 × 133 × 30 and × 25	

5.4 Case I: Columns braced in both directions — simple construction

For simple multistorey construction braced in both directions the columns should be designed by applying nominal moments only at the beam-to-column connections. The following conditions should be met:

(a) columns should be effectively continuous at their splices
(b) pattern loading may be ignored
(c) all beams framing into the columns are assumed to be fully loaded
(d) nominal moments are applied to the columns about the two axis
(e) nominal moments may be proportioned between the length above and below the beam connection according to the stiffness's I/L of each length, except that when the ratio of the stiffnesses does not exceed 1.5 the moment may be divided equally
(f) nominal moments may be assumed to have no effects at the levels above and below the level at which they are applied
(g) the equivalent uniform moment factor m and the slenderness correction factor n should both be taken as unity
(h) the slenderness λ of the columns should not exceed 180.

Notes to simple construction method:
1. The nominal moments as calculated in subclause 5.4.1 (*d*) are the minimum moments to be used for column design.
2. When actual (other than nominal) moments are applied to the columns by eccentrically connected beams, cantilevers or by a full frame analysis then the column design should be carried out using the Case II method as described in subsection 5.5.

5.4.1 Design procedure

(a) Calculate the factored beam reactions = 1.6 × imposed load + 1.4 × dead load from the beams bearing onto the column from each axis at the level considered. It may also be necessary to calculate the reactions for different load factors for different load combinations.

(b) Calculate the factored axial load F on the column at level being considered.

(c) Choose a section for the lowest column length from the following:

203 UC for buildings up to 3 storeys high
254 UC for buildings up to 5 storeys high
305 UC for buildings up to 8 storeys high
356 UC for buildings from 8 to 12 storeys high

If UC sections are not acceptable choose a UB section from Table 16. In that case the column design procedure described in clause 5.4.2 should be followed.

(d) Calculate the nominal moments applied to the column about the two axes by multiplying the factored beam reactions by eccentricities based on the assumption that the loads act at the face of the column + 100mm, or at the centre of a stiff bearing, whichever is greater.
If the beam is supported on a cap plate the load should be taken as acting at the edge of the column or edge of any packing.

(e) Obtain the nominal moments M_x and M_y applied to each length of the column above and below the beam connections by proportioning the total applied nominal moments, from *(d)* according to the rule stated in clause 5.4 *(e)*.

(f) Choose a trial section and grade of steel such that the following equation is satisfied:

$$\frac{F_c}{A_g p_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y Z_y} < 1$$

where F_c is factored axial load on the column
p_c is the compressive strength
A_g is the gross cross-sectional area
M_x is the nominal moment about the major axis
M_y is the nominal moment about the minor axis
M_{bs} is the buckling resistance moment for simple columns
Z_y is the elastic modulus about the minor axis
p_y is the design strength obtained from Table 2

(g) When the slenderness of the column on the y-y axis is 0.85 × the storey height the values of the capacities $A_g\, p_c$ and of the buckling resistance moments M_{bs} and $p_y\, Z_y$ may be obtained from the Tables in Appendix C for UC sections. Alternatively, these may be obtained by calculation as described in clauses 5.4.2 and 5.4.3 or from the blue book.

(h) When the above values have been obtained the equation in *(f)* should be checked. If the relationship shown is not satisfied a larger section or higher grade of steel should be chosen and the calculations repeated.

The section finally chosen may be used for the full height of the building. Alternatively, the calculations should be repeated at the splice levels, which are normally at 2- or 3-storey intervals above the lowest floor if it is considered desirable to reduce the sections at upper levels.

5.4.2 Alternative design procedure for calculation of compressive resistance P_c for columns

As an alternative procedure to that described in subclause 5.4.1 *(g)* the compressive resistance P_c of a column may be obtained from:

$P_c = A_g \times p_c$

where A_g is the gross sectional area of the trial section, and

p_c is the compressive strength.

The alternative procedure is to:

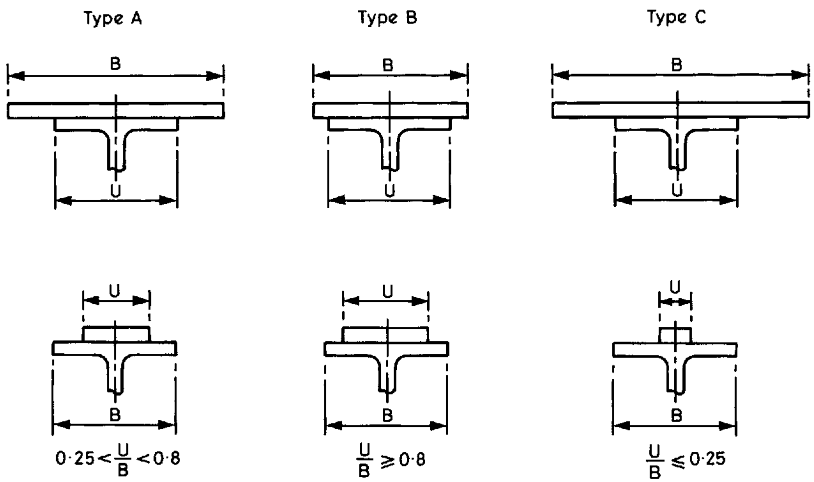
(a) choose a trial section avoiding slender UB sections and obtain the design strength p_y from Table 2 according to the thickness of the flanges and grade of steel of the chosen section

(b) calculate the slenderness λ by dividing the effective length L_E obtained as in subsection 5.2 by the radius of gyration of the chosen section about the relevant axis.

(c) determine p_c from the graphs in Appendix D according to the type of section, axis of buckling, slenderness and the appropriate design strength p_y; for intermediate values of p_y interpolation may be used.
To interpret these tables is should be noted that the definitions of I- and H-sections are as follows:

I-sections have a central web and two equal flanges, with an overall depth greater than 1.2 times the width of the flanges
H-sections have a central web and two equal flanges, with an overall depth not greater than 1.2 times the width of the flanges.

(d) calculate the compressive resistance P_c from A_g p_c.
The graphs in Appendix D may also be used for the design of compound columns using the appropriate plate type as shown in Fig. 5.



5 Plate types for compound sections

5.4.3 Alternative procedure for calculation of M_{bs} the buckling resistance moments for columns for simple construction

Calculate M_{bs} the buckling resistance moment capacity about the major axis from the equation

$$M_{bs} = S_x p_b$$

where S_x is the plastic modulus of the section about the major axis, and
p_b is the bending strength obtained from Table 10 using an equivalent slenderness
$\lambda_{LT} = 0.5 \times L/r_y$
where L is the distance between levels at which both axes are restrained in position, and
r_y is the radius of gyration about the minor axis.

5.5 Case II: Columns braced in both directions subject to applied moments other than nominal moments

Design procedure

(a) Calculate the factored axial load F on the column at the level being considered $= 1.6 \times$ imposed load $+ 1.4 \times$ dead load. It may also be necessary to calculate the axial load using different load factors for different load combinations.

(b) Calculate the factored moments M_x and M_y on the major and minor axis, respectively, from the most unfavourable combination of dead and imposed loads using the load factors and load combinations from Table 1.

(c) Calculate the ratios β of the moments applied about both axes at each end of the column, and then determine the equivalent uniform moment factors m_x and m_y from Table 9.

(d) Choose a trial section avoiding 'slender' UB sections, and then carry out checks for local capacity and overall buckling. If the equivalent uniform moment factors m_x and m_y are taken as, or are equal to, 1.0 then the local-capacity check need not be carried out.

5.5.1 Local capacity check

This should be carried out at the locations of the greatest bending moment and axial load (usually at the ends) by checking that :

$$\frac{F}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} < 1$$

The procedure to be followed is:

(a) determine the design strength p_y from Table 2 according to the grade of steel and the flange thickness.

(b) calculate $F/A_g p_y$

(c) calculate the b/t ratio for the flange outstand and the d/t ratio for the web where b is the width of the flange outstand
d is the depth of the web
t is the thickness of the element concerned.

If the b/t ratio exceeds 15ε or the d/t ratio exceeds 39ε, where $\varepsilon = (275/p_y)^{1/2}$

then elements of the trial section may be slender and a further trial section should be chosen.

Alternatively, the strength reductions necessary for slender sections should be calculated by reference to BS 5950.

(d) if a new section has been chosen to satisfy the requirements for flange outstand b/t and limiting web d/t, check p_y and then recalculate $F/A_g p_y$

(e) obtain the moment capacities M_{cx} and M_{cy} in the absence of axial load from the blue book about the major and minor axis, respectively, and then calculate

$$\frac{M_x}{M_{cx}} \text{ and } \frac{M_y}{M_{cy}}$$

(f) finally, check that $\dfrac{F}{A_g p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} < 1{\cdot}0$.

5.5.2 Overall buckling check

This should be carried out by checking that:

$$\frac{F_c}{A_g p_c} + \frac{\overline{M}_x}{M_{bx}} + \frac{\overline{M}_y}{p_y Z_t} < 1$$

where F_c is the compressive force due to axial load
p_c is the compression resistance obtained as in clause 5.4.2
A_g is the gross cross-sectional area
$\overline{M}_x$ is the equivalent uniform moment about the major axis
M_{bx} is the buckling resistance moment M_b obtained as in subsection 4.4 or 4.5, as appropriate, but should not be taken as greater than $p_y Z_x$
$\overline{M}_y$ is the equivalent uniform moment about the minor axis
p_y is the design strength obtained from Table 2
Z_y is the elastic section modulus about the minor axis
Z_x is the elastic section modulus about the major axis.

The equivalent uniform moments $\overline{M}_x$ and $\overline{M}_y$ should be obtained by multiplying the factored moments M_x and M_y obtained as in clause 5.5(*b*) by the equivalent uniform moment factors m determined as follows:

for $\overline{M}_x$: m is the greater of m_{LT} and m_x
for $\overline{M}_y$: $m = m_y$
where m_{LT} is the value of m obtained from Table 9 based on the pattern of major axis moments over the length between restraints to the minor axis
m_x is the value of m obtained from Table 9 based on the pattern of major axis moments over the length between restraints to the major axis
m_y is the value of m obtained from Table 9 based on the pattern of minor axis moments over the length between restraints to the minor axis.

5.6 Cased columns

5.6.1 Introduction

This subsection describes the design of cased columns that are subject to compression and bending. To allow for the additional stiffening afforded by the concrete casing, the casing should comply with the conditions set out in clause 4.6.2. for cased beams.

5.6.2. Design procedure

A trial section should be chosen and calculations made to satisfy the following equations:

(a) For local capacity:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} < 1$$

(b) For overall buckling resistance:

$$\frac{F_c}{P_c} + \frac{m_{LT}M_x}{M_b} + \frac{m_yM_y}{M_{cy}} < 1$$

where $F_c, M_x, M_y, M_b, M_{cx}, M_{cy}, m_{LT}$ and m_y are as calculated for the uncased section, and

$$P_{cs} = \left(A_g + 0{\cdot}25\,\frac{f_{cu}}{p_y}\,A_c\right)p_y$$

where A_c is the gross sectional area of the concrete but neglecting any casing in excess of 75mm from the overall dimensions of the steel section and neglecting any applied finish

A_g is the gross sectional area of the steel strut

f_{cu} is the characteristic concrete cube strength at 28 days of the encasement but < 40 N/mm^2

p_y is the design strength of the steel section for the flange thickness and grade of steel

$$p_c = (A_g + 0{\cdot}45\,\frac{f_{cu}A_c}{p_y})\,p_c \text{ but not greater than } P_{cs}$$

where p_c is the compressive strength of the steel sections calculated using either

$$\lambda = \left(\frac{L_E}{0{\cdot}2b_c}\right) \text{ but} \not> \left\{\frac{L_E}{0{\cdot}2(B+150)}\right\}, \text{ or}$$

$$\lambda = L_E/r_x$$

b_c is the minimum width of solid casing

B is the overall width of the flange, and

r_x is the radius of gyration of the steel section alone about its major axis.

6 Braced multistorey buildings — bracing and other members

6.1 Introduction

This Section describes the design of bracing and other members that are subject to compression or tension only, and bending combined with compression or tension.

In all cases the value of the slenderness λ obtained by dividing the effective length L_E by the radius of gyration about the relevant axis should not exceed the following:

(a)	for members resisting loads other than wind loads	180
(b)	for members resisting self-weight and wind loads only	250
(c)	for any member normally acting as a tie but subject to reversal of stress resulting from the action of wind	350

6.2 Bracing members in compression only

For members in compression only, the compressive resistance P_c should be obtained in accordance with the design procedure in clause 5.4.2. For discontinuous struts the eccentricities arising from the connections may be ignored, and members shown in Table 17 may be designed as axially loaded only.

The slenderness should be obtained from Table 17, where the Length L should be taken as the distance between the intersection of centroidal axes or the intersections of the setting out lines of the bolts, and r is the radius of gyration about the relevant axis defined in Table 17.

It should be noted that Table D3 in Appendix D should be used to obtain p_c for rolled angles, channels, T-sections and laced and battened sections.

6.3 Bracing members in compression and bending with moments other than those due to connection eccentricities

Members in combined bending and compression should be designed in accordance with the procedure in subsection 5.5. UBs, UCs RSJs and RSCs, CHSs or RHSs should be chosen as indicated in Tables 6 or 16 in order to avoid the use of slender elements. For angles and T-sections in order to avoid slender sections, the b/t ratio should not exceed 15ε, and for the stems of T-sections the d/t ratio should not exceed 19ε

where $\varepsilon = \left(\dfrac{275}{p_y}\right)^{1/2}$

6.4 Bracing members in tension only

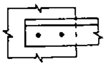
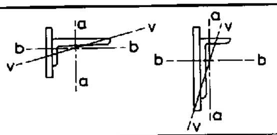
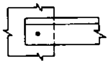
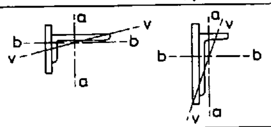
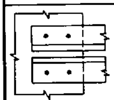
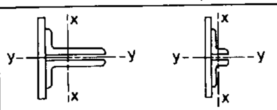
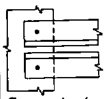
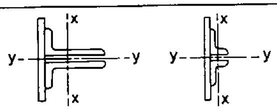
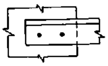
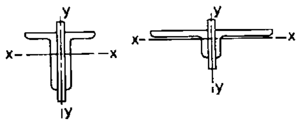
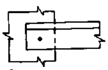
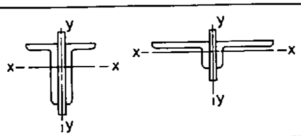
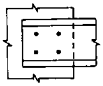
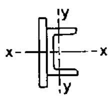
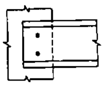
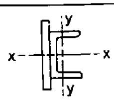
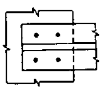
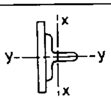
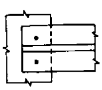
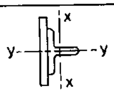
The tension capacity of a member should be calculated from

$$p_t = A_e p_y$$

where A_e = the effective area of the member, and

p_y = the design strength obtained from Table 2 according to the grade of steel and thickness of the flange.

Table 17 Slenderness for discontinuous angle, channel and T-section struts

Connection	Sections and axes	Slenderness ratios (see notes 1 and 2)
		vv axis: $0{\cdot}85L_{vv}/r_{vv}$ but $\geq 0{\cdot}7L_{vv}/r_{vv}+15$ aa axis: $1{\cdot}0L_{aa}/r_{aa}$ but $\geq 0{\cdot}7L_{aa}/r_{aa}+30$ bb axis: $0{\cdot}85L_{bb}/r_{bb}$ but $\geq 0{\cdot}7L_{bb}/r_{bb}+30$
 See note 3		vv axis: $1{\cdot}0L_{vv}/r_{vv}$ but $\geq 0{\cdot}7L_{vv}/r_{vv}+15$ aa axis: $1{\cdot}0L_{aa}/r_{aa}$ but $\geq 0{\cdot}7L_{aa}/r_{aa}+30$ bb axis: $1{\cdot}0L_{bb}/r_{bb}$ but $\geq 0{\cdot}7L_{bb}/r_{bb}+30$ (see note 3)
 See note 4		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $((0{\cdot}85L_{yy}/r_{yy})^2+\lambda_c^2)^{1/2}$ but $\geq 1{\cdot}4\lambda_c$ (see note 5)
 See note 4		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $((L_{yy}/r_{yy})^2+\lambda_c^2)^{1/2}$ but $\geq 1{\cdot}4\lambda_c$ (see note 5)
 See note 4		xx axis: $0{\cdot}85L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $((L_{yy}/r_{yy})^2+\lambda_c^2)^{1/2}$ but $\geq 1{\cdot}4\lambda_c$ (see note 5)
 See notes 3 and 4		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $((L_{yy}/r_{yy})^2+\lambda_c^2)^{1/2}$ but $\geq 1{\cdot}4\lambda_c$ (see notes 3 and 5)
		xx axis: $0{\cdot}85L_{xx}/r_{xx}$ yy axis: $1{\cdot}0L_{yy}/r_{yy}$ but $\geq 0{\cdot}7L_{yy}/r_{yy}+30$
		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ yy axis: $1{\cdot}0L_{yy}/r_{yy}$ but $\geq 0{\cdot}7L_{yy}/r_{yy}+30$
		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $0{\cdot}85L_{yy}/r_{yy}$
		xx axis: $1{\cdot}0L_{xx}/r_{xx}$ but $\geq 0{\cdot}7L_{xx}/r_{xx}+30$ yy axis: $1{\cdot}0L_{yy}/r_{yy}$

Notes to Table 17 are on page 49

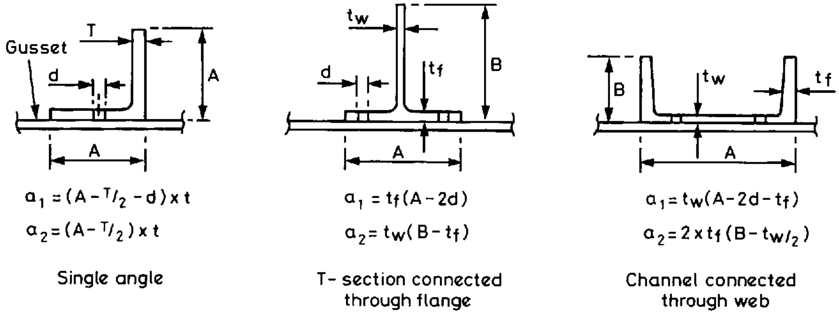
6.4.1 Angles, channels and T-sections

In simple tension members composed of angles, channels, and T-sections any eccentricity may be ignored, and the members may be treated as axially loaded provided that the effective areas, A_e are taken as follows:

(a) *Single angles connected through one leg only, channels connected through web only, or T-sections connected through flange only*

$$A_e = (a_1 + a_2) \times \frac{3a_1}{3a_1 + a_2}$$

where a_1 = net area of connected leg, web or flange
a_2 = gross area of unconnected legs or flanges as illustrated in Fig. 6.



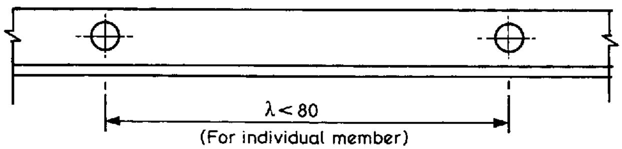
6 *Illustrations for areas* a_1 *and* a_2

(b) *Double-angle ties*

Where back-to-back double angles are connected to *one* side of a gusset plate or section and are:

(i) in contact or separated by packing plates not greater in thickness than the sum of the unconnected legs, and

(ii) connected by bolts such that the slenderness of each angle does not exceed 80 as shown in Fig. 7



7 *Slenderness of individual member*

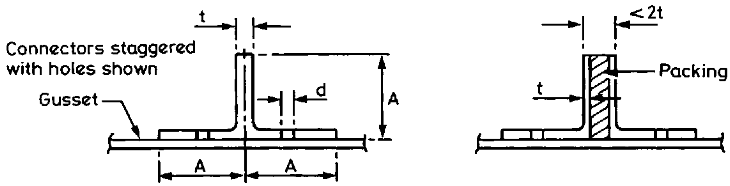
Notes to Table 17

1. The length L is taken between the intersections of the centroidal axes or the intersections of the setting-out lines of the bolts, irrespective of whether the strut is connected to a gusset or directly to another member.
2. Intermediate lateral restraints reduce the value of L for buckling about the relevant axes. For single angle members, L_{vv} is taken between lateral restraints perpendicular to either aa or bb.
3. For single or double angles connected by one bolt, the compression resistance is also reduced to 80% of that for an axially loaded member see BS 5950, subclauses 4.7.10.2(b) and 4.7.10.3(d).
4. Double angles are either battened (see BS 5950, clause 4.7.12) or interconnected back-to-back (see BS 5950, clause 4.7.13). Battens or interconnecting fasteners are also needed at the ends of members.
5. $\lambda_c L_{vv}/r_{vv}$ = with L_{vv} measured between interconnecting fasteners for back-to-back struts or between end welds or, end fasteners of adjacent battens or battened angle struts.

then the effective area, A_e, may be taken as:

$$A_e = (a_1 + a_2) \times \frac{5a_1}{5a_1 + a_2}$$

where a_1 = net area of connected legs, and
a_2 = gross area of unconnected legs as illustrated in Fig. 8.



$a_1 = 2(A - t/2 - d) \times t$
$a_2 = 2(A - t/2) \times t$

8 Illustrations for areas a_1 *and* a_2

(c) *Single angles connected through both legs, single channels connected through both flanges, single or T-sections connected through leg only or through leg and flange, and internal bays of continuous ties*
For these types of connection and member the effective area, A_e, should be taken as A_e = net area of the section, where the total net area is calculated using the equations for a_1 and a_2 in Fig. 6.

(d) *Double-angle ties connected to both sides of a gusset or section*
Provided that the following requirements are met:

(i) the angles are connected by bolts or welds and are held apart by packing pieces in at least two places
(ii) the outermost of the connections in (i) should be at a distance from the end connection of approximately 9 times the shortest leg dimension, and
(iii) the connecting bolts should be the same diameter as those used for the end connection.

then A_e = net area of the section as illustrated in Fig. 9.

6.4.2 Other sections

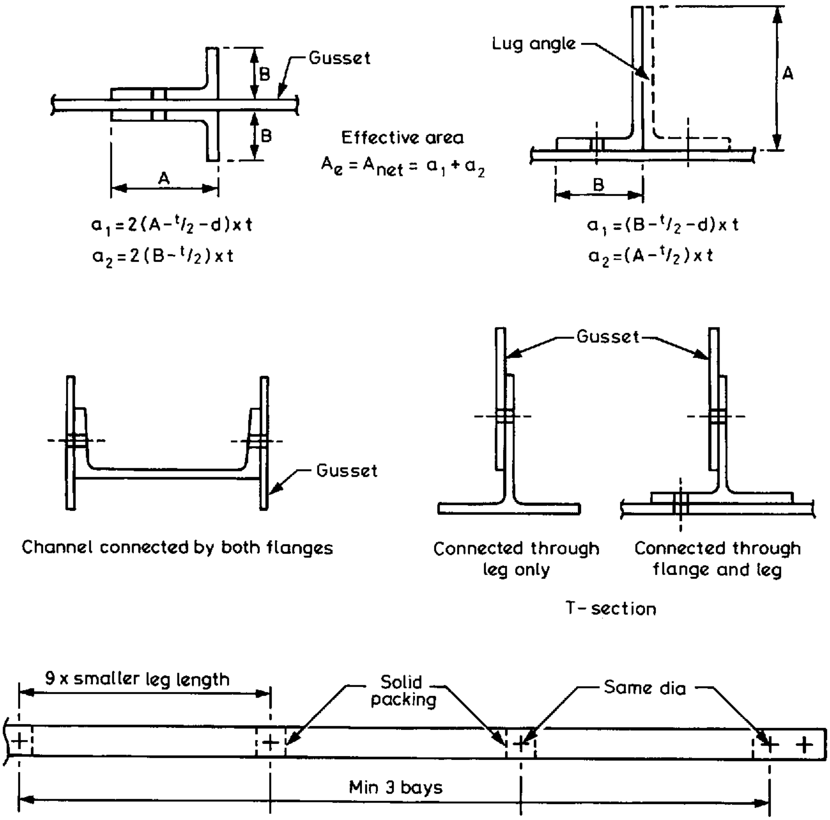
The tension capacity of sections, other than those given in clause 6.4.1, may be found from:

$$P_t = A_e p_y$$

where, A_e, the effective area is found multiplying the net area at a connection by the factor K_e, which for BS 4360 steels can be taken as

K_e = 1.2 for grade 43 steel
K_e = 1.1 for grade 50 steel

In no case should the effective area A_e be taken as greater than the gross area A_g of the section under consideration.



9 Net areas of double-angle ties

6.5 Bracing member in tension and bending

Tension members should be checked for capacity at the points of greatest bending moments and axial loads, usually at the ends. The following relationship should be satisfied:

$$\frac{F}{A_e p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} < 1$$

where F is the factored axial load in member
A_e is the effective area as defined in subsection 6.4
p_y is the design strength obtained from Table 2
M_x is the factored moment about the major axis at critical region
M_y is the factored moment about the minor axis at critical region
M_{cx} is the moment capacity about the major axis in the absence of axial load obtained from the blue book
M_{cy} is the moment capacity about the minor axis in the absence of axial load obtained from the blue book.

These members should also be checked for bending strength only as given in Section 4.

7 Braced multistorey buildings — robustness

Multistorey construction that has been framed in accordance with the recommendations given in clause 3.2.4 and designed in accordance with the rest of the *Manual*, should produce a robust construction subject to the connections also being designed in accordance with the *Manual.*

However in order to demonstrate that the requirements for robustness are met, the following checks should be carried out:

1. Beams, ties and their connections at each column and beam in two orthogonal directions at each floor and roof should be checked for the following factored tensile forces which need not be considered as additive to other loads:

   internal ties $0{\cdot}5\ W_f\ S_t\ L_a$
   edge ties $0{\cdot}25\ W_f\ S_t\ L_a$

   where W_f is the total factored dead and imposed load per unit area
   S_t is the mean transverse spacing of the ties, and
   L_a is the greatest distance in the direction of the tie or beam, between adjacent lines of columns or vertical supports.

   The factored tie forces noted above should not be taken as less than 75 kN for floors, 40 kN at roofs, and for perimeter columns 1% of the factored vertical load at the level of the ties or beams, and may also be used for multistorey buildings of less than 5 storeys.

For composite and *in situ* floors the reinforcement in the floor construction may be used to provide the tying forces.

For the purposes of provision 1 it may be assumed that substantial deformation of members and their connections is acceptable.

2. Column splices should be checked for a tensile force of not less than two-thirds of the factored vertical load applied to the column from the next floor level below the splice.
3. Any beam that carries a column together with the members that support it should be checked for localization of damage using the following alternative procedures:

   (a) Area of damage, following the notional removal at each storey in turn of a single column or beam supporting a column, should be assessed. Failure should be contained within the adjacent storeys or restricted to an area within those storeys of approximately either 70m² or 15% of the area of the storey, whichever is less.

   When carrying out the check the following should be noted:

   (i) It may be assumed that substantial permanent deformation of members and their connections is acceptable, e.g. the loads can be carried by catenary action
   (ii) Except in buildings used predominately for storage or where the imposed loads are of permanent nature, one-third of the normal imposed loads should be used. Also, only one-third of the normal wind loads need be considered
   (iii) Partial safety factor for loads, γ_f, should be taken as 1.05 except that γ_f of 0.9 should be used for dead loads restoring overturning action.

(b) Where damage cannot be localized, using the method in clause 7.3 (*a*), the relevant members should be designed as follows:

(i) Stipulated accidental load (normally taken as $34kN/m^2$) should be applied to the member in question, from the appropriate direction. The reaction on the member from other components attached to the member should be calculated on the basis of the same accidental load but should be limited to the ultimate strength of these components or their connections
(ii) In conjunction with the above the effects of ordinary loads should be considered using partial safety factors noted in subclause 7.3(*a*) (iii) above
(iii) Any structural component vital to the stability of the relevant members as defined in clause 7.3(*b*) should itself be designed in a similar way to that described in clause 7.3(*b*).
(iv) Subclauses 7.3(*a*)(i), (ii) and (iii) apply.

8 Braced multistorey buildings — the next step

8.1 Introduction

When the design of the structural steel members has been completed, preliminary general-arrangement drawings (including sections through the entire structure) and, if necessary, typical connection details should be prepared and sent to other members of the design team for comment, together with a brief statement of the principal design assumptions, i.e. imposed loadings, weights of finishes, fire ratings and durability.

8.2 Connections

It is important to establish the general form and type of connections assumed in the design of the members and to check that they are practicable. It is also important to consider the location of edge beams and splices, the method and sequence of erection of steelwork, access and identification of any special problems and their effects on connections such as splicing/connection of steelwork erected against existing walls. The extent of welding should also be decided, as it may have an effect on cost.

Design of typical connections in preliminary form is necessary when:

- appearance of exposed steelwork is critical
- primary and secondary stresses occur that may have a direct influence on the sizing of the members
- connections are likely to affect finishes such as splices affecting column casing sizes and ceiling voids
- steelwork is connected to reinforced concrete or masonry, when greater constructional tolerances may be required, which can affect the size and appearance of the connections
- unusual geometry or arrangement of members occurs
- holding bolts and foundation details are required
- a detail is highly repetitive and can thus critically affect the cost.

When necessary the design of connections should be carried out in accordance with Sections 14 and 15.

8.3 Finalization of design

Before the design of the structure can be finalized, it is necessary to obtain approval of the preliminary drawings from the other members of the design team. The drawings may require further amendment, and it may be necessary to repeat this process until approval is given by all parties. When all the comments have been received, it is then important to marshall all the information received into a logical format ready for use in the final design. This may be carried out in the following sequence:

- checking all information
- preparation of a list of design data
- amendments of drawings as a basis for final calculations.

8.4 Checking all information

The comments and any other information received from the client and the members of the design team and the results of the ground investigation should be checked to verify that the design assumptions are still valid.

Stability
Check that no amendments have been made to the sizes and to the disposition of the bracing to the structure. Check that any openings in these can be accommodated in the final design.

Movement joints
Check that no amendments have been made to the disposition of the movement joints.

Loading
Check that the loading assumptions are still correct. This applies to dead and imposed loading such as floor finishes, ceilings, services, partitions, and external wall thicknesses, materials and finishes thereto. Verify the design wind loading, and consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account.

Fire resistance, durability and sound insulation
Confirm with other members of the design team the fire resistance required for each part of the structure, the corrosion protection that applies to each exposure condition and the mass of floors and walls (including finishes) required for sound insulation.

Foundations
Examine the information from the ground investigation and decide on the type of foundation to be used in the final design. Consider especially any existing or future structure adjacent to the perimeter of the structure that may influence not only the location of the foundations but also any possible effect on the superstructure and on adjacent buildings.

Performance criteria
Confirm which codes of practice and other design criteria are to be used in the design.

Materials
Confirm the grade of steel and type of bolts or welds to be used in the final design for each or all parts of the structure. The use of different grades of bolts and different types of welds on the same structure should be avoided.

Erection
Review the effect on the design of the method and sequence of erection and the use of any temporary erection bracing.

8.5 Preparation of design data list
The information obtained from the above check and that resulting from any discussions with the client, design team members, building control authorities and material suppliers should be entered into a design information data list. A suitable format for such a list is included in Appendix E. This list should be sent to the design team leader for approval before the final design is commenced.

8.6 Amendment of drawings as a basis for final calculations
The preliminary drawings should be brought up to date, incorporating any amendment arising out of the final check of the information previously accumulated and finally

approved. In addition, the following details should be added to all the preliminary drawings as an aid to the final calculations:

Grid lines

Establish grid lines in two directions, approximately at right-angles to each other. Identify these on the plans.

Members

Give all slabs, beams and columns unique reference numbers or a combination of letters and numbers related if possible to the grid, so that they can be readily identified on the drawings and in the calculations.

Loading

Mark on the preliminary drawings the loads that are to be carried by each slab. It is also desirable to mark on the plans the width and location of any walls or other special loads to be carried by the slabs or beams.

8.7 Sequence for finalizing design

When all the above checks, design information, data lists and preparation of the preliminary drawings have been carried out, the design of the structure should be finally checked. This should be carried out in the same logical sequence, as in the preceeding sections, e.g.:

floors
beams
columns
bracing and other members
robustness
connections.

The redesign of any steel members that may be necessary should be carried out as described for each member in the preceeding sections.

9 Single-storey buildings – general

9.1 Introduction

This Section offers advice on the general principles to be applied when preparing a scheme for a single-storey structure. The aim should be to establish a structural scheme that is practicable, sensibly economic, and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Loads should be carried to the foundation by the shortest and most direct routes. In constructional terms, simplicity implies (among other matters) repetition, avoidance of congested, awkward or structurally sensitive details, with straightforward temporary works and minimal requirements for unorthodox sequencing to achieve the intended behaviour of the completed structure.

Sizing of structural members should be based on the longest spans (sheeting rails and purlins) and largest areas of roof (frames and foundations). The same sections should be assumed for similar but less onerous cases. This saves design and costing time and is of actual advantage in producing visual and constructional repetition and hence, ultimately, cost benefits.

Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good 'benchmark'.

Scheme drawings should be prepared for discussion and budgeting purposes, incorporating such items as general arrangement of the structure including bracing, type of roof and wall cladding, beam and column sizes, and typical edge details, critical and unusual connection details, and proposals for fire and corrosion protection.

When the comments of the other members of the design team have been received and assimilated, the structural scheme should be revised and the structural members redesigned as necessary.

9.2 Loads

Loads should be based on BS 648[3], BS 6399:Parts 1 and 3[4] and CP3: Chapter V: Part 2[5] (or BS 6399 Part 2[4], in preparation).

9.2.1 Imposed loading, Q_k

Imposed loading is specified in BS 6399: Part 3 as a minimum of 0.60 kN/m^2 for pitched roofs of 30° to the horizontal or less. For these roofs where access, other than for normal maintenance, is required, the minimum imposed load should be increased to 1.5 kN/m^2. BS 6399: Part 3 also gives the loads arising from the effects of uniformly distributed and drifted snow.

9.2.2 Wind loading, W_k

Wind loading is specified in CP3: Chapter V: Part 2 or BS 6399: Part 2 (in preparation) and varies with the roof pitch and the location of the building.

9.2.3 Dead and service loading, G_k

Dead loading consists of the weight of the roof sheeting, and equipment fixed to the roof, the structural steelwork, the ceiling and any services. The following loads may be used in the absence of firm details.

- roof sheeting and side cladding — 0·1 to 0·2 kN/m^2
- steelwork — 0·1 to 0·2 kN/m^2
- ceiling and services — 0·1 to 0·3 kN/m^2

9.2.4 Notional loading, N_k

Notional horizontal load, N_k at each level should be the greater of:

$1\% \times 1{\cdot}4\ G_k$ or $0{\cdot}5\% \times (1{\cdot}4\ G_k + 1{\cdot}6\ Q_k)$

where G_k and Q_k are the unfactored loads from the level considered.

9.2.5 Strength and stability limit states

The load combinations and load factors to be used in design for the limit states of strength and stability are shown in Table 1 (repeated here for convenience). The factored loads to be used for each load combination should be obtained by multiplying the unfactored loads by the appropriate load factor γ_f from Table 1.

Table 1 Load combinations and load factors γ_f

load combination	load type					
	dead, G_k		imposed, Q_k		wind, W_k	notional, N_k
	adverse	beneficial	adverse	beneficial		
1 dead + imposed	1·4	1·0	1·6	0		
2 dead + wind	1·4	1·0			1·4	
3 dead + wind + imposed	1·2	1·0	1·2	0	1·2	
4 dead + imposed + notional horizontal	1·4		1·6			1·0

The 'adverse' and 'beneficial' factors should be used so as to produce the most onerous condition. When appropriate, temperature effects should be considered with load combinations 1, 2 and 3.

9.3 Material selection

In the UK, grade 43 steel should be used generally for rafters and columns, although grade 50 may be used unless deflection is likely to be critical. Grade 50 steel may be used for latticed or trussed members. Grade 8.8 bolts should normally be used throughout.

9.4 Structural form and framing

The most common forms of single-storey frames are:

portal frames with pinned bases
posts with pinned bases and pitched trusses
posts with pinned bases and nominally parallel lattice girders.

These alternatives may be provided with fixed bases, but this is not generally adopted since it may result in the provision of uneconomic foundations.

The design of the framing should be based on:

(a) The following spacings between frames and spans, which are likely to be economic:

	spacing, m	span, m
portals	5·0 – 7·0	up to 30
post-and-pitched truss	4·5 – 7·5	18·0 – 25
post-and-lattice girder	4·5 – 7·5	20·0 – 40

(b) For portal frames provide longitudinal stability against horizontal forces by placing vertical bracing in the side walls deployed symmetrically wherever possible

(c) For post-and-truss or post-and-lattice girder frames provide stability against lateral forces in two directions approximately at right-angles to each other by arranging suitably braced bays deployed symmetrically wherever possible.

(d) Provide bracing in the roof plane of all single-storey construction to transfer horizontal loads to the vertical bracing

(e) Provide bracing to the bottom of members of trusses or lattice girders if needed to cater for reversal of forces in these members because of wind uplift

(f) Consider the provision of movement joints for buildings in the UK whose plan dimensions exceed 50 m.

(g) Purlins should, where possible, be supported at node points for lattice girders and trusses. Alternatively, the effects of local bending at the lattice members need to be taken into account

(h) For post and truss frames the trusses should be supported on the columns at the intersection point of the rafter and tie members of the truss

(i) The arrangement should take account of openings for doors and windows and support for services and problems with foundations, e.g. columns immediately adjacent to site boundaries may require balanced or other special foundations.

(j) Provide framing to openings to transfer horizontal forces to braced elements.

Finally, a choice has to be made between portal frames, post-and-truss, post-and-lattice girder frames. This should be based on the required appearance of the structure, on the extent of support required in the roof for ceilings and services, and on overall economy.

9.5 Fire resistance

Fire protection should be considered for those frames that provide lateral stability to perimeter or party walls and which are required to have a fire rating. This can be achieved by fire casing either the whole frame, or only the stanchions. In the latter case, the stability of the stanchion should be justified as a cantilever in fire conditions. Fire protection should also be considered for structural members supporting mezzanine floors enclosed by single-storey buildings.

9.6 Corrosion protection

Structural steelwork should be protected from corrosion. For different parts of the steelwork in a single-storey building this may be achieved as follows:

(a) *Steelwork integral with external cladding and that which is not readily accessible for inspection and maintenance*
Concrete encasement, or
high-quality corrosion protection (i.e. galvanizing and bituminous paint, etc).

(b) *Internal steelwork*
A protective system commensurate with the internal environment.

(c) *External steelwork*
A protective system commensurate with the external environment.

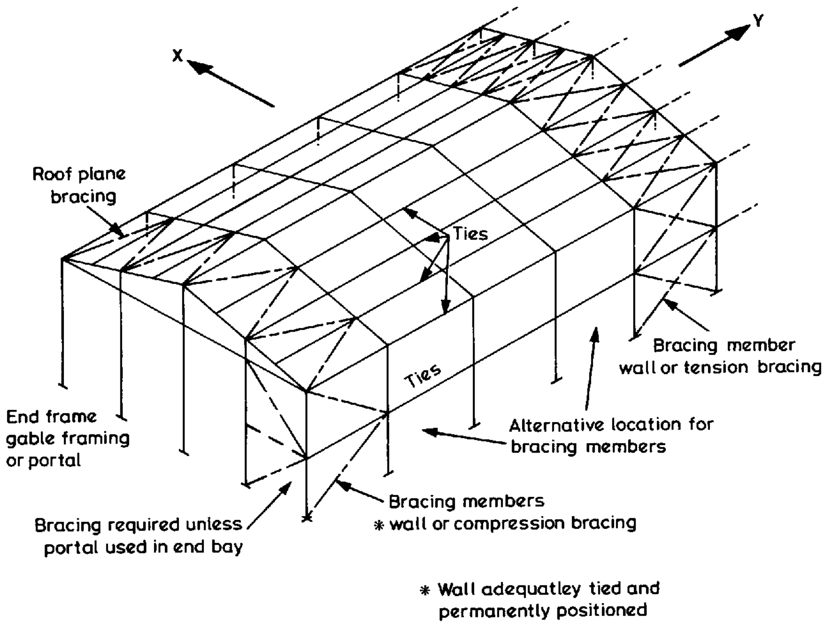
For different environments and for more detailed advice reference should be made to subsection 3.6, to BS 5493[11] and to pamphlets published by BSC, BCSA, ZDA and Paintmakers Association.

The preparation of steel surfaces prior to painting has a crucial effect on the life of the paint system and should be carefully specified.

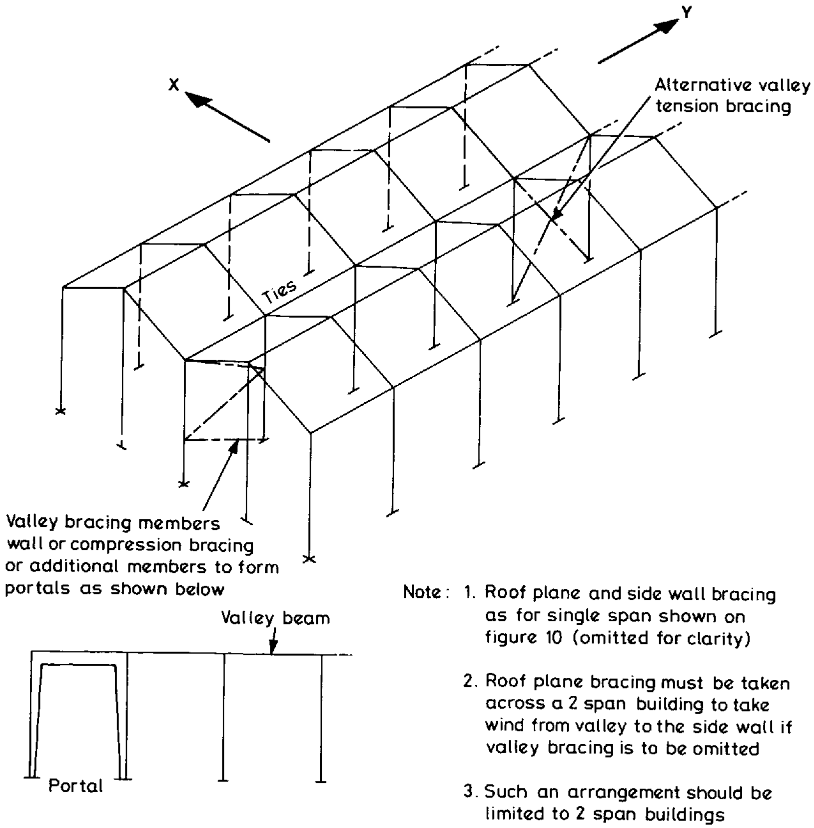
9.7 Bracing

Choose the location and form of bracing in accordance with the recommendations in subsection 9.4 and clause 2.2.3. Typical locations are shown on Figs. 10 and 11 for single-storey buildings.

Wind loads on the structure should be assessed for the appropriate load combinations and divided into the number of bracing bays resisting the horizontal forces in each direction.



10 Single-span portals



11 Multispan portals

9.8 Roof and wall cladding

Although this *Manual* is concerned with the design of structural steelwork, it is essential at the start of the design to consider the details of the roof and cladding systems to be used, since these have a significant effect on the design of steelwork.

The choice of cladding material largely depends on whether the roof is flat or pitched. For the purposes of this *Manual*, a roof will be considered flat if the roof pitch is less than 6°. It should be noted, however, that roofs with pitches between 6° and 10° will often require special laps and seals to avoid problems with wind-driven rain etc.

The variety of materials available for pitched roofs is vast and cannot possibly be dealt with in detail in a *Manual* such as this. However, a brief description of the most common forms is included in Table 18 which summarizes the salient features of the various types of lightweight roofing systems commonly used in the UK for single-storey structures.

Table 18 Lightweight roofing systems and their relative merits

description	minimum pitch	typical depth mm	typical span mm	degree of lateral restraint to supports	comments
galvanized corrugated steel sheets	10°	75 sinusoidal profile	1800 to 2500	good if fixed direct to purlins	low-budget industrial and agricultural buildings limited design life, not normally used with insulated liner system
fibre-cement sheeting	10°	25 to 88	925 to 1800	fair	low-budget industrial and agricultural buildings brittle construction usually fixed to purlins with hook bolts
profiled aluminium sheeting (insulated or uninsulated)	6°	30 to 65	1200 to 3500	good if fixed direct to purlins	good corrosion resistance but check fire requirements & bi-metallic corrosion with mild-steel supporting members
profiled coated steel sheeting (insulated or uninsulated)	6°	25 to 65	1500 to 4500	good if fixed direct to purlins	the most popular form of lightweight roof cladding used for industrial type buildings wide range of manufactures, profile types & finishes
'standing seam' roof sheeting (steel or aluminium)	2°	45	1100 to 2200	no restraint afforded by cladding, clip fixings	used for low-pitch roof and has few or no laps in direction of fall usually requires secondary supports or decking, which may restrain main purlins
galvanized steel or aluminium decking systems	nominally flat	32 to 100	1700 to 6000	very good	used for flat roof with insulation and vapour barrier and waterproof membrane over fire and bimetallic corrosion to be checked if aluminium deck used
timber	nominally flat	general guidance as for timber floors			
reinforced woodwool slabs	nominally flat	50 to 150	2200 to 5800	good if positively fixed to beam flanges	pre-screeded type can prevent woodwool becoming saturated during construction

In general, most of the profiled decking systems described in Table 18 for pitched roofs are available for use as wall cladding. Where insulation is required this can be provided either as bonded to the sheeting or in a 'dry-lining' form with the internal lining fixed to the inside face of the sheeting rails. Similarly, fire protection of the walls of industrial buildings can be achieved by using boarding with fire-resistant properties on the inside face of the sheeting rails.

It is not uncommon to provide brickwork as cladding for the lower 2.0 – 2.5m of industrial buildings, since all profiled sheeting is easily damaged. Where this detail is required it is usually necessary to provide a horizontal steel member at the top of the wall spanning between stanchions to support such brick panel walls against lateral loading.

10 Single-storey buildings — purlins and side rails

10.1 Purlins

Purlins may consist of cold-formed, rolled or hollow sections.

10.1.1 Cold-formed sections

For cold-formed sections empirical rules and design formulae are given in BS 5950: Part 5[20]. However, the section sizes for cold-formed purlins can be determined from the safe load tables in the technical literature provided by the manufacturers of cold-formed members for use as purlins. Anti-sag bars tied across the apex should be provided as recommended by the manufacturers.

10.1.2 Angles and hollow sections

Angles and hollow section purlins may be designed in accordance with the empirical method, provided that they comply with the following rules:

- claddings and fixing thereof to be capable of providing lateral restraints to the purlins
- grade of steel to be 43
- unfactored loads to be considered, and loading to be basically uniformly distributed
- imposed load used in design to be not less than 0.75 kN/m^2
- span not to exceed 6.5m, and roof pitch not to excced 30°
- purlins to be connected at each end by at least two fixings.

If these rules cannot be complied with then the purlins should be designed as beams. If the rules are complied with then the elastic modulus, Z, and dimensions D and B, depth and width, respectively, should not be less than the following:

section	Z,cm^3	D,mm	B,mm
angles	$\frac{WL}{1800}$	$L/45$	$L/60$
CHS	$\frac{WL}{2000}$	$L/65$	—
RHS	$\frac{WL}{1800}$	$L/70$	$L/150$

where W is the total unfactored load in kN due to either (dead plus imposed) or (wind less dead), and
L is the span in mm.

10.2 Side rails

Side rails may consist of angles, hollow sections or cold-formed sections.

10.2.1 Cold-formed sections

The section sizes for cold-formed side rails can be determined from the safe load tables in the technical literature provided by the manufacturers of cold-formed members for use as side rails. Anti-sag rods tied to an eaves beam should be provided as recommended by the manufacturers.

10.2.2 Angles and hollow sections

Angle and hollow-section side rails may be designed by the empirical method provided that they comply with the following rules:

- claddings and fixing thereto to be capable of providing lateral restraint to the side rails
- grade of steel to be 43
- unfactored loads to be considered and the loading to consist of wind and self-weight of cladding only
- span not to exceed 6.5m
- side rails to be connected at each end by at least two fixings.

If these rules cannot be complied with then the side rails should be designed as beams. If they are complied with then the elastic modulii Z_1 and Z_2 of the axes parallel and perpendicular to the plane of the cladding, respectively, and the dimensions D and B perpendicular and parallel to the plane of the cladding should not be less than the following:

section	Z_1,cm³	Z_2,cm³	D,mm	B,mm
angles	$\frac{W_1L}{1800}$	$\frac{W_2L}{1200}$	$L/45$	$L/60$
CHS	$\frac{W_1L}{2000}$	$\frac{W_2L}{1350}$	$L/65$	—
RHS	$\frac{W_1L}{1800}$	$\frac{W_2L}{1200}$	$L/70$	$L/70$

where W_1 and W_2 are the unfactored loads in kN acting perpendicular and parallel, respectively, to the plane of the cladding, and
L is the span in mm.

(Note: For Z_2 and B, L may be taken as the span between anti-sag rods provided that these are properly supported.)

11 Portal frames with pinned bases

11.1 Elastic design

Elastic frame analysis may be used to obtain the loads and moments on the frame. The members should then be designed using the procedures given in Sections 4 and 5 for the design of beams and columns. In addition, sway stability checks should be carried out as for plastic design.

11.2 Plastic design

Plastic methods of analysis are commonly used in the design of portal frames. Guidance is given on the design of single-span portals with pinned bases and where wind loading does not control the design.

For multibay frames of equal spans where the same rafter section is used throughout, the design is almost invariably governed by that for external bays. The internal stanchions are subjected to very little bending unless the loading is asymmetrical, and may therefore conservatively be sized the same as the stanchions for the single-span case.

It should however be noted that the eaves deflections of pitched multibay frames should be carefully checked, as the horizontal deflections will be cumulative.

If it is necessary to refine the design for multibay frames and for frames where wind is likely to govern the design, then specialist literature should be consulted and/or computer programs used.

The procedure for the plastic method of design of portal frames with pinned bases is given in this *Manual* in the following sequence:

- sizing of rafters and stanchions
- check on sway and snap-through stability
- check on serviceability — deflection
- check on position of plastic hinge and calculation of load capacity of frame
- check on stability of plastic hinges, rafter, haunch and leg.

11.3 Single-storey portals — sizing of rafters and stanchions

The plastic design of a portal with pinned bases is carried out in this *Manual* by the selection of members from graphs. This method is based on the following assumptions:

(a) plastic hinges are formed at the bottom of the haunch in the stanchion and near the apex in the rafter, the exact position being determined by the frame geometry

(b) the depth of the haunch below the rafter is approximately the same as the depth of the rafter

(c) the haunch length is not more than 10% of the span of the frame, an amount generally regarded as providing a balance between economy and stability

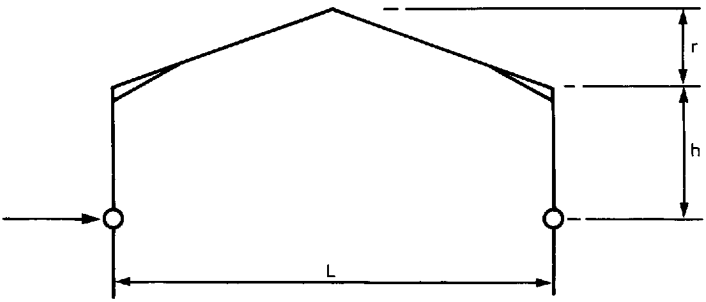
(d) the moment in the rafter at the top of the haunch is $0.87M_p$, and it is assumed that the haunch region remains elastic

(e) the calculated values of M_p are provided exactly by the sections and that there are no stability problems. Clearly these conditions will not always be met and the chosen sections should be fully checked for all aspects

(f) wind loading does not control design.

Design procedure

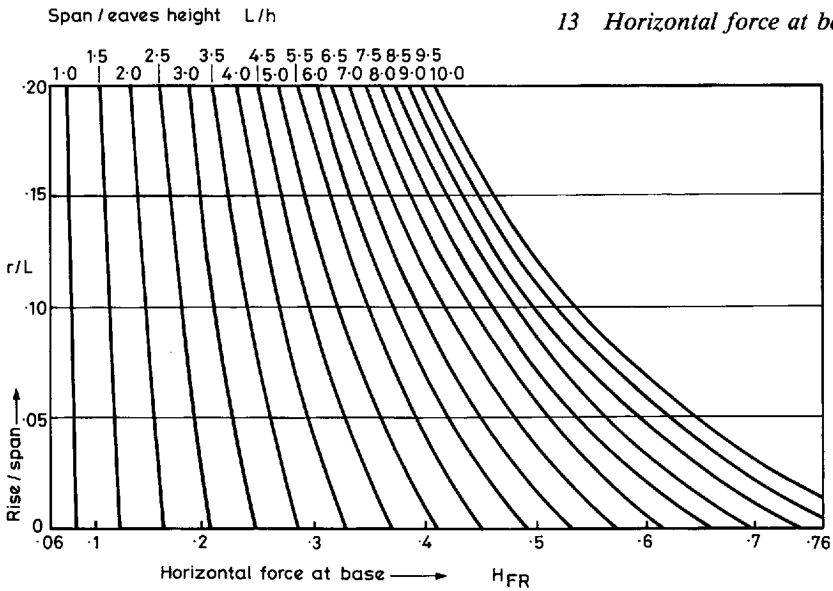
The procedure to be adopted is set out below, and the various dimensions are shown on Fig. 12.

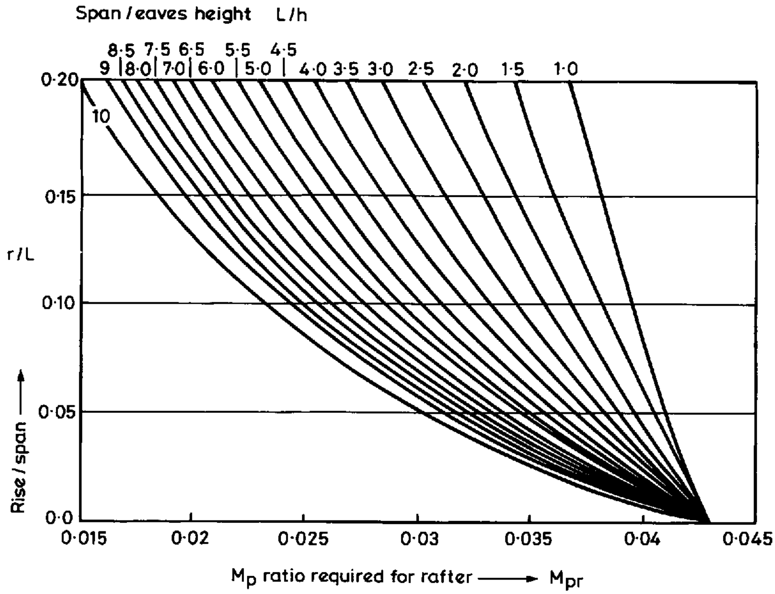


12 Dimensions of portal

- *(a)* calculate the span/height to eaves ratio $= L/h$
- *(b)* calculate the rise/span ratio $= r/L$
- *(c)* calculate the total factored load WL on the frame from subsection 9.2, and then calculate WL^2, where W is the load per unit length of span L (e.g. $W = ws$, where w is the total factored load per m² and s is the bay spacing)
- *(d)* from Fig. 13 obtain the horizontal force ratio H_{FR} at the base from r/L and L/h
- *(e)* calculate the horizontal force at base of span $H = H_{FR}\ WL$

13 Horizontal force at base





14 M_p *ratio required for rafter* M_{pr}

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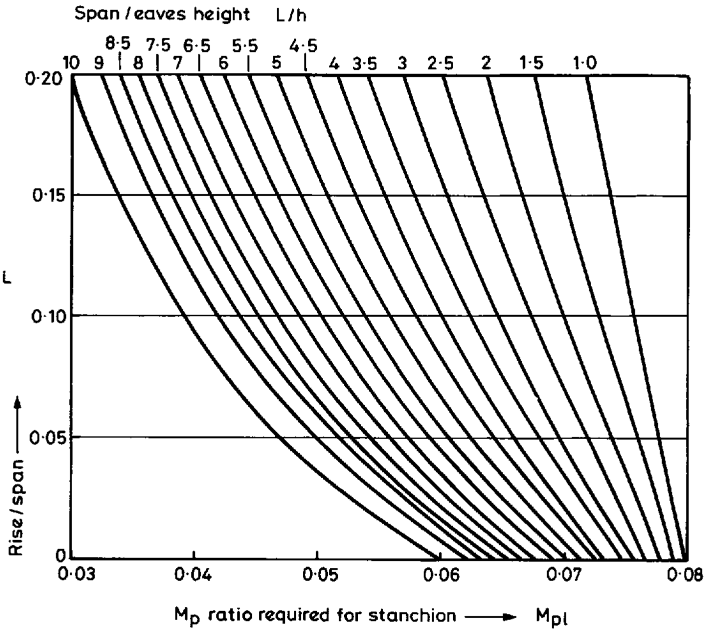
15 M_p *ratio required for stanchion* M_{pl}



(f) from Fig. 14 obtain the rafter M_p ratio M_{PR} from r/L and L/h
(g) calculate the M_p required in the rafter from M_p (rafter) $= M_{pr}\ WL^2$
(h) from Fig. 15 obtain the stanchion M_p ratio M_{pl} from r/L and r/h
(i) calculate the M_p required in the stanchion from M_p (stanchion) $= M_{pl}\ WL^2$
(j) determine the plastic modulii for the rafter S_{XR} and stanchion S_{XL} from
$S_{XR} = M_p$ (rafter)$/p_y$
$S_{XL} = M_p$ (stanchion)$/p_y$

where p_y is the design strength obtained from Table 2.

Using these plastic modulii, the rafter and stanchion sections may be chosen from the range of plastic sections as so defined in the blue book.

11.4 Sway and snap-through stability

Two modes of failure have been identified for portal frames, the first may occur in any frame and is called 'sway stability'. The mode of failure is caused by the change in frame geometry due to applied loading which gives rise to the $P\Delta$ effect, when axial loads on compression members displaced from their normal positions give moments that reduce the frame's capacity.

The second mode can take place when the rafter frames of 3 or more bays have their sections reduced because full advantage has been taken of the fixity provided in the valleys. In this case the risk of 'snap-through' should be considered.

The *Manual* gives equations so that both of these cases can be checked. If either check is not satisfied then it is essential that the member sizes are adjusted so that the frame passes these checks. If access to more elaborate methods of calculation is available then the deflection of the frame against certain criteria may be used as a less conservative approach.

11.4.1 Sway stability check

This should be carried out for all frames by checking that the ratio of L_b/D is equal to or less than that obtained from the following equation:

$$\frac{L_b}{D} < \frac{44}{\Omega} \cdot \frac{L}{h} \cdot \frac{(\quad \varrho \quad)}{(4 + \varrho L_r/L)} \cdot \frac{(275)}{(P_{yr})}$$

in which L_b is the effective span of the bay. When the depth of the haunches from top of rafter to underside of haunch is not less than $2D$ then $L_b = L - L_h$, otherwise L_b should be taken as equal to L,

where $\varrho = (2I_c/I_r).(L/h)$ for single-bay frames or
$\varrho = (I_c/I_r).(L/h)$ for multibay frames
L = span of frame
L_h = haunch length; if the haunches at each side of the bay are different, the mean value should be taken
D = minimum depth of rafters
h = stanchion height
I_c = moment of inertia of column = 0 if it is not rigidly joined to rafter
I_r = moment of inertia of rafter at its shallowest point
P_{yr} = design strength of rafter (p_y)
L_r = developed length of rafter $= 2\sqrt{\{(L/2)^2 + r^2\}}$

$\Omega = W_r / W_o$ ratio of the arching effect of the frame
where W_r = factored vertical load on rafters
W_o = maximum uniformly distributed vertical load that can be carried by the rafter treated as a fixed end beam of span $L = 16\ S_{XR}\ P_{yr}/L$, where S_{XR} = plastic modulus of the rafter about the x-axis. This formula can be used only where the load on the rafter is substantially uniformly distributed.

If the condition given in this formula is satisfied then the frame will remain stable under loading, and deflections will not seriously affect strength. If the condition is not satisfied, then the frame members sizes must be adjusted so that the (L_b/D) condition is satisfied. It is still important that movements of the frame and their effect on cladding, finishes and appearance are checked.

11.4.2 Snap-through stability check

This should be carried out for frames of 3 or more spans, as in each internal bay snap-through instability may occur because of the spread of the stanchions and inversion of the rafter.

To prevent this the rafter slenderness should be such that:

$$\frac{L_b}{D} < \frac{22(4 + L/h)}{\Omega(\Omega - 1)} \left(1 + \frac{I_c}{I_r}\right) \frac{(275)}{(P_{yr})} \tan 2\theta_r$$

where θ_r for the symmetrical ridged frame is the rafter slope.

For any other roof shape

$$\theta_r = \tan^{-1}\left(\frac{2h_1}{L}\right)$$

where h_1 is the height of the apex above the top of the stanchions.
No limit need be placed on L_b/D when $\Omega < 1$ and the other symbols are as defined in clause 11.4.1.

11.5 Serviceability check — deflection

The horizontal deflection at the eaves may be estimated for unfactored loads by obtaining the deflection factor D from Fig. 16 using L/h and the angle of the roof slope θ.

The estimated horizontal deflection of one side stanchion at the eaves, d_E, is then obtained from

$$d_E = D\left(\frac{hL}{d_r} \cdot \frac{p_y}{g_p}\right) 10^{-6}$$

where h = height to eaves in mm
L = span in mm
d_r = depth of rafter in mm
p_y = design strength in N/mm^2, and
g_p = the load factor on the frame, which may be taken as 1.5 for this check.

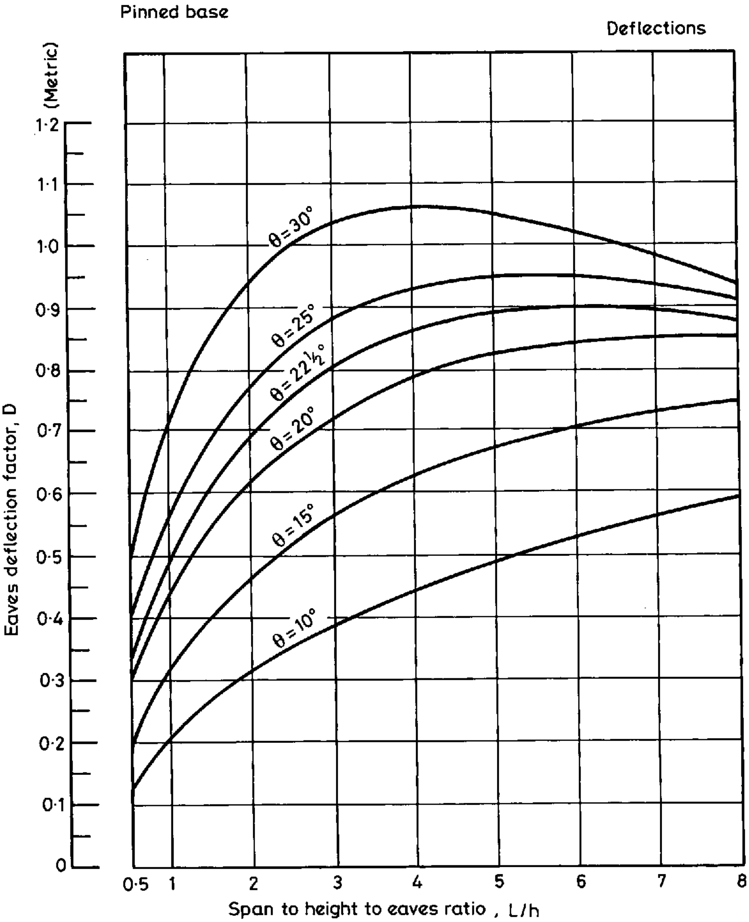
This horizontal deflection d_E should not normally exceed $h/300$ as indicated in clause 2.6.1 unless claddings are used which can accommodate larger deflections.
The deflection of the ridge should be obtained from

$$d_{RE} = d_E \times \cot\theta$$

and should be small enough to have no adverse affect the cladding, finishes and appearance.

Member sizes should be adjusted if either d_E or d_{RE} exceeds the chosen permissible values.

It should be noted that more accurate estimates of the deflections may be obtained by the use of suitable computer programs.



16 Deflections at unfactored loads — pinned-base frames

11.6 Check on position of plastic hinge in rafter and calculation of load capacity

In order to check that the correct mode of failure has been assumed a reactant diagram should be drawn. This is obtained by plotting the moments due to the applied forces and known moments at hinge locations, including feet. If the moments at all points in the frame are less than the values of M_p and only equal to M_p at the hinge locations then the assumptions may be considered as satisfactory. If M_p of the frame is exceeded at any point in the frame then the diagram must be adjusted to take this into account.

In order to check the position of the plastic hinge and the load capacity of the frame previously designed the following simple procedure may be carried out:

(a) consider a pinned based portal frame subject to vertical loading as shown in Fig. 17

(b) calculate $H = M_p$ (stanchion)$/h_1$

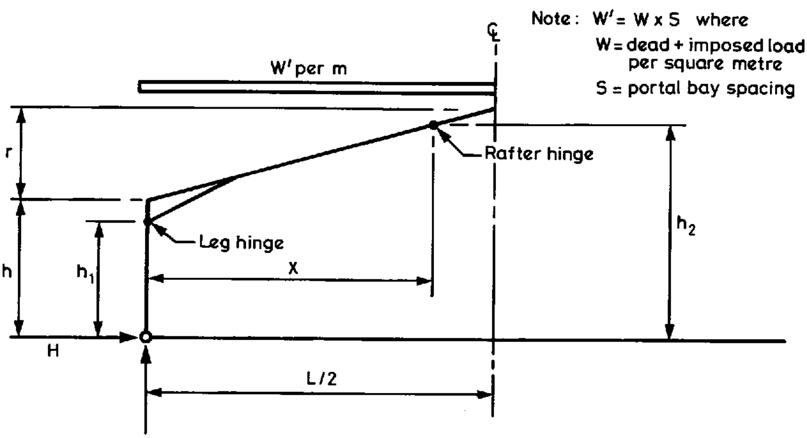
(c) take moments about the rafter hinge position giving

$$M_p \text{ (rafter)} = W'L/2\ X - H\ h_2 - W'X^2/2$$

(d) calculate r/L and L/h and then determine X from Fig. 18.

(e) calculate W' from the equation in (3)

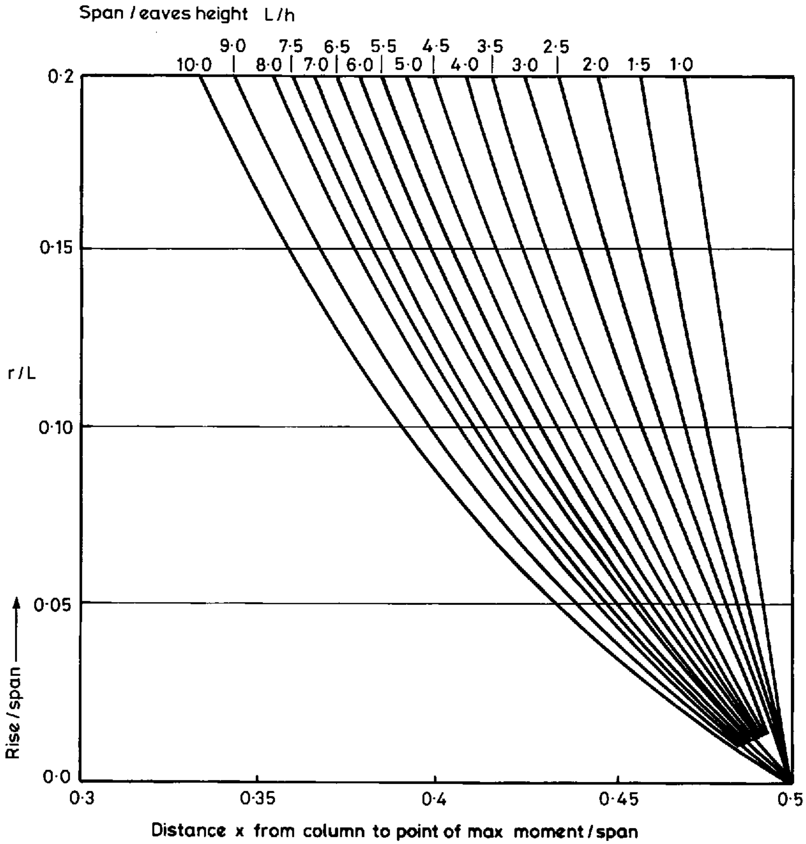
(f) redesign the frame if the load capacity W' is less than the total factored load on the frame.



17 Vertically loaded pinned-base portal frame

11.7 Stability checks

The following stability checks should be carried out:
- restraint of plastic hinges
- stability of rafter
- stability of haunch
- stability of stanchion.



18 Distance x *from column to point of maximum moment/span*

11.7.1 Restraint of plastic hinges

(a) A restraint should be provided to both flanges at each plastic hinge location. If this is not practicable, the restraint should be provided within a distance of half the depth of the member along the flanges of the member from the location of the plastic hinges.

(b) The maximum distance L_m in m from the hinge restraint to the next adjacent restraint should not exceed

$$L_m = \frac{38\, r_y}{\sqrt{\left\{\dfrac{f_c}{130} + \left(\dfrac{p_y}{275}\right)^2 \left(\dfrac{x}{36}\right)^2\right\}}}$$

where f_c is the average compression stress due to the axial load (in N/mm^2)
p_y is the design strength (in N/mm^2) from Table 2
r_y is the radius of gyration in mm about the minor axis
x is the torsional index.

Where the member has unequal flanges, r_y should be taken as the lesser of the values for the compression flange only or the whole section.

Where the cross-section of the member varies within the length L_m the minimum value of r_y and the maximum value of x should be used.

(c) If the member is restrained on the tension flange then the maximum distance to the nearest restraint on the compression flange may be taken as L_t calculated as for the stability of haunch (see clause 11.7.3).

11.7.2 Rafter stability

The rafter should be checked to see that stability is maintained in all load cases. Unless there is wind uplift, the following checks should be made:

(a) the plastic hinge location as obtained in subsection 11.6 near the ridge should be restrained for the case of a uniform load

(b) a purlin or other restraint is needed on the compression flange at a distance L_m calculated from the plastic hinge restraint formula given in clause 11.7.1

(c) further restraints to the top flange are required so that the rafter satisfies the requirements of Conditions III and IV in subsections 4.4 and 4.5 for beams without full lateral restraint. These conditions will automatically be satisfied if the purlin restraints are at spacings less than those obtained from the slenderness λ_{Lo} in Table 11a

(d) in areas where there is compression on the bottom flange the procedure given for haunches in clause 11.7.3 should be applied using constants applicable to haunch/depth of rafter = 1.

In cases where there is wind reversal the rafters should be checked as in subclause 11.7.2 *(d)*.

11.7.3 Stability of haunch

Provided that the tension flange of the haunch is restrained, then the maximum length between restraint to the compression flange of the haunch should be limited to the L_t obtained as shown below, provided that:

(a) the rafter is a UB section

(b) the haunch flange is not smaller than the rafter flange

(c) the depth of the haunch is not greater than 3 times the depth of the rafter

(d) the buckling resistance is satisfactory if it is checked as though it were a compression flange in accordance with subsections 4.4 or 4.5 using an effective length L_E equal to the spacing of the tension flange restraints.

L_t may conservatively be taken as:

$$\frac{K_1 r_y x}{\sqrt{(72x^2 - 10^4)}} \text{ for grade 43 steel}$$

$$\frac{K_2 r_y x Y}{\sqrt{(94x^2 - 10^4)}} \text{ for grade 50 steel}$$

where r_y is the minimum radius of gyration of the rafter section.
x is the torsional index of the rafter section.
K_1 and K_2 have the following values:

depth of haunch/depth of rafter
= 1: $K_1 = 620$ $K_2 = 645$
= 2: $K_1 = 495$ $K_2 = 515$
= 3: $K_1 = 445$ $K_2 = 465$

If no restraint is provided to the tension flange then the limiting length L_m to the nearest restraint on the compression flange should be calculated as for restraint of plastic hinges.

11.7.4 Stability of stanchion

Near the top of the stanchion a restraint should be provided at the location of the plastic hinge, together with a further restraint at a distance L_m below the position of the hinge restraint.

If the stanchion is restrained on the tension flange as described in subclause 11.7.3(*d*) then the distance to the nearest restraint on the compression flange may be taken as L_t as calculated for the stability of the haunch.

The stanchion should then be checked in accordance with the overall buckling check in subsection 5.5 to see if a further compression flange restraint is required.

This restraint should be provided if found necessary using the side rails. Side rails may be positioned to suit the cladding if no further compressive restraint is required.

12 Lattice girder or truss with pin-based columns

12.1 Lattice girders or trusses

These members should be designed using the following criteria:

(a) connections between web and chord members may be assumed to be pinned for calculation of axial forces in the members

(b) members meeting at a node should be arranged so that their centroidal axis (or lines of bolt groups) coincide. When this is not possible the members should be designed to resist the resulting bending moments caused by the eccentricities of connections in addition to the axial forces
Similarly, bending moments arising from loading between node points (other than self-weight) should be taken into account

(c) fixity of connections and rigidity of members may be taken into account for calculating the effective lengths of the members

(d) secondary stress in the chord and web members may be ignored provided that:

- the loads are applied at the node points
- length/depth ratios of the chord members in the plane of the girder or truss are greater than 12
- length/depth ratios of the web members in the plane of the girder or truss are greater than 24

(e) the length of chord members may be taken as the distance between the connections to the web members in the plane of the girder or truss and the distance between the longitudinal ties or purlins in the plane of the roof cladding.

(f) ties to chords should be properly connected to an adequate restraint system

(g) bottom members should be checked for load reversal due to wind uplift.

The procedure to be adopted to size the members of lattice girders or trusses is set out below.

Calculate the total factored load on the roof from subsection 9.2.

Determine the forces in the members of the lattice girder or truss for all relevant load combinations by graphical methods, drawing a force diagram, by the method of sections, resolution of forces at a joint, or a computer analysis.

The section sizes may then be obtained as follows:

compression members
calculate the effective lengths L_E using Table 15 or 17. Design the section by reference to clause 5.4.2 or by reference to the compressive resistance of members in the tables in the blue book as appropriate

tension members
design the section by reference to subsection 6.4 or to the tension capacity tables in the blue book.

deflection
the deflection of girder and trusses should be checked to see that serviceability with particular reference to roof drainage is not impaired.

12.2 Columns for single-storey buildings braced in both directions

Design procedure

(a) Calculate the factored axial load, F, on the column from the roof, and from the side cladding

(b) Calculate the factored wind loading on the side walls and on the roof

(c) Calculate the factored horizontal component, W_R, of the wind force on the roof for use in the design of bracing members

(d) Calculate the total factored side wall wind loads, W_{W1} and W_{W2}, on the external columns

(e) Calculate the maximum factored moments arising from wind on the columns from M = (greater of W_{W1} or W_{W2}) x $h/8$
where h is the height of the column from base to eaves

(f) Calculate the factored nominal moments on the columns arising from the imposed and dead load by assuming a nominal eccentricity as for multistorey columns or by elastic analysis and add these to the factored wind moments

(g) Select a section and check the design of the column as for Case II in subsection 5.5.

12.3 Columns for single-storey buildings braced in one direction only in the side walls and/or in the valleys

Design Procedure

(a) Calculate the factored axial load, F, on the column from the roof, and from the side cladding

(b) Calculate the factored wind loading on the side walls and on the roof

(c) Calculate the factored horizontal component, W_R, of the wind force on the roof

(d) Calculate the total factored sidewall wind loads, W_{W1} and W_{W2} on the external columns

(e) Calculate the maximum factored moments on the columns arising from wind and dead and imposed loads by elastic analysis assuming that the columns and lattice girders or trusses act as frames in the unbraced direction

(f) Select a section and check the design of the column as for Case II in subsection 5.5.

13 Single-storey buildings — other members etc.

13.1 Gable posts

Calculate the factored axial loads and factored wind moments on these posts. Select a section and check design as for Case II in subsection 5.5.

13.2 Bracing and tie members

Assess the appropriate factored wind load on the bracing and tying members in each braced bay, and then design the members in accordance with the methods described in Section 6.

13.3 Other members

It may be necessary to provide framing for door, window and services opening in the sidewalls of the single-storey building. These members should be sized in accordance with the methods recommended above for gable posts or bracing members, depending on the loading in or location of the member.

13.4 The next step

Preliminary general arrangement drawings should be prepared when the design of the structural members has been completed, and sent to other members of the design team for comments.

It is important to establish the general form and type of connections assumed in the design of the members and to check that they are practicable. Reference should be made to subsection 8.2 and Section 15 as the items described therein also apply to single-storey buildings

The details to be shown, checking of information, preparation of a list of design data, the finalization of the design, etc. should be carried as described in Section 8 for multistorey buildings.

14 Connections

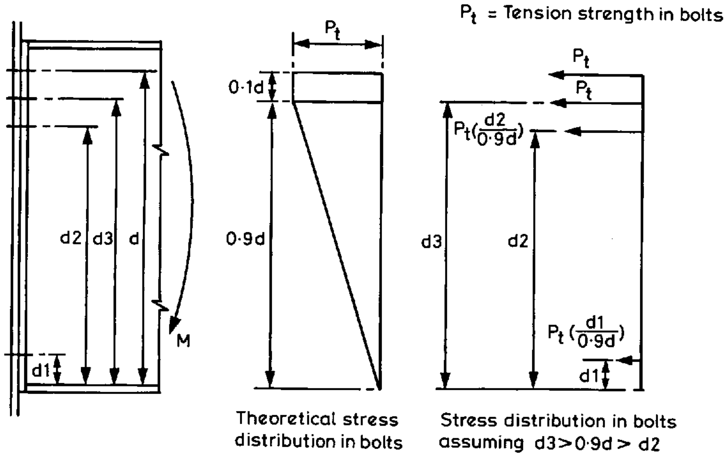
14.1 General

Connections may be designed on the basis of a realistic assumption of the distribution of internal forces, provided that they are in equilibrium with the externally applied loads. The analysis of the forces on the connections, which can be either elastic or plastic, should be carried out using factored forces and moments, noting the following:

(a) The centroidal axes of the connected members should meet at a point; otherwise the effect of the eccentricity of the connection should be taken into account in the design of the members

(b) Generally the bolts and welds in a splice connection should be designed to carry all the forces. Where the ends of the members have machined bearings, compressive forces may be transferred by direct contact

(c) In bolted moment connections, the stress distribution shown in Fig. 19 may be used



19 Stress distribution on bolts

(d) As far as possible only one size and grade of bolts should be used on a project. Bolts should generally be of 8.8 grade and not less than 12 mm in diameter and weld sizes should not be less than 5mm

(e) The local ability of the connected members to transfer the applied forces should be checked and stiffeners provided where necessary

(f) Bolts should generally be sheradized, spun galvanized or otherwise treated to be compatible with the paint protection system for the steel frame

(g) Where dissimilar metals are likely to be in contact in a moist environment, suitable isolators such as neoprene washers and sleeves should be incorporated to prevent bimetallic corrosion

(h) Locking devices or high-strength friction-grip bolts may be incorporated in connections subjected to vibration impact, or when slip is unacceptable.

It is particularly important to check the connections on site. Identification marks for commonly used bolts and nuts are shown in Appendix F.

Critically loaded welded connections should be tested using non-destructive methods such as ultrasonic tests to BS 3923[22] for butt welds and magnetic particle inspection to BS 6072[23] for fillet welds.

14.2 Bolts

14.2.1 Spacing and edge distances

A summary of the requirements is given in Table 19.

Table 19

requirement	distance
minimum spacing	$2 \cdot 5d$
maximum spacing in unstiffened plate:	
in direction of stress in any environment	$14t$
exposed to corrosion in any direction	$16t < 200$mm
minimum edge and end distance:	
rolled, machine flame cut or planed edge	$1 \cdot 25D$
sheared or hand flame cut edge	$1 \cdot 4D$
any end in the direction that the fastener bears	$1 \cdot 4D$
maximum edge distance:	
normal	$11t\varepsilon$
exposed to corrosive influences	40mm + 4t

In Table 19 t is the thickness of the thinner part
d is the nominal bolt diameter
D is the hole diameter
ε is $\sqrt{(275/p_y)}$

14.2.2 Strength checks

The strength of ordinary bolts to carry the forces should be checked using the formulae in Table 20

Table 20 Strength check for bolts

required strength	formula
shear capacity	$P_s = p_s A_s$
bearing capacity of bolt	$P_{bb} = dtp_{bb}$
bearing capacity of ply	$P_{bs} = dtp_{bs} < 1/2etp_{bs}$
long joints where distance between the first and last rows of bolts in the direction of the load exceeds 500 mm - shear capacity	$P_s = p_s A_s \left(\frac{5500 - L_j}{5000}\right)$
large grips where the total thickness of the connected plies exceeds 5 × the nominal diameter d of the bolts - shear capacity	$P_s = p_s A_s \left(\frac{8d}{3d + T_g}\right)$
tension capacity	$P_t = p_t A_t$
combined shear and tension	$\frac{F_s}{P_s} + \frac{F_t}{P_t} < 1{\cdot}4$ but no part should be greater than 1·0

In Table 20 p_s is the shear strength obtained from Table 21
p_{bb} is the bearing strength of the bolt obtained from Table 21
p_{bs} is the bearing strength of the ply obtained from Table 22
e is the end distance but not greater than $2d$
p_t is the tension strength of the bolt
A_s is the effective area for shear
A_t is the tensile stress area
L_j is the length of the joint (mm)
T_g is the thickness of the grip (mm)
d is the nominal bolt diameter
t is the thickness of the ply

Table 21 Values of p_s, p_{bb}, p_t

	bolt grade		other grades of fasteners N/mm²
	grade 4.6 N/mm²	grade 8.8 N/mm²	
shear strength, p_s	160	375	$0{\cdot}48U_f$ but $\leqslant 0{\cdot}69\ Y_f$
bearing strength, p_{bb}	460	1035	$0{\cdot}72\ (U_f + Y_f)$
tension strength, p_t	195	450	$0{\cdot}58\ U_f$ but $\leqslant 0{\cdot}83\ Y_f$

Y_f is the specified minimum yield strength of the fastener.
U_f is the specified minimum ultimate tensile strength of the fastener.

Table 22 Values of p_{bs}, p_{bg}

	steel to BS 4360 grade 43 N/mm²	steel to BS 4360 grade 50 N/mm²	other grades of steel N/mm²
p_{bs}	460	550	$0{\cdot}65(U_s + Y_s)$
p_{bg}	825	1065	$2{\cdot}2U_s$ but $\leqslant 3{\cdot}0\ Y_s$

Y_s is the specified minimum yield strength of the steel.
U_s is its specified minimum ultimate tensile strength.

Table 23 Minimmum shank tension P_o (proof load)

bolt diameter, mm	proof load, kN
M12	49·4
M16	92·1
M20	144·0
M24	207·0
M30	286·0

For hsfg bolts the formula in Table 24 should be used.

Table 24 Strength check for hsfg bolts

required strength	formula
slip resistance (parallel shank)	$P_{sl} = 1{\cdot}1K_s\mu P_o$
bearing capacity	$P_{bg} = dtp_{bg} < 1/3\ etp_{bg}$
slip resistance — (long joints)	$P_{sl} = 0{\cdot}6P_o\left(\dfrac{5500 - L_j}{5000}\right)$
slip resistance (waisted shank)	$P_{sl} = 0{\cdot}9K_s\mu P_o$
tension capacity	$P_t = 0{\cdot}9P_o$
combined shear and tension	$\dfrac{F_s}{P_{sl}} + 0{\cdot}8\dfrac{F_t}{P_t} <1$ but F_s/P_{s1} and F_t/P_t should both be less than 1

In Table 24 P_o is the minimum shank tension obtained from Table 23
K_s is 1·0 for clearance holes
0·85 for short slotted holes *
0·6 for long slotted holes *
μ is the slip factor < 0·45

* For definition and use reference should be made to BS 5950.

p_{bg} is the bearing strength of connected parts obtained from Table 22
F_s is the applied shear
F_t is the externally applied tension

The capacity of bolts are also tabulated in the blue book.

14.3 Welds

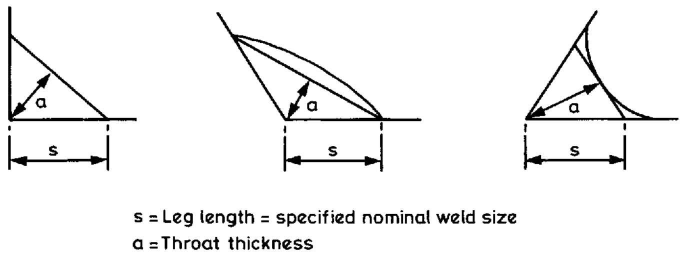
14.3.1 Fillet welds

Fillet welds are designed using an effective throat thickness *a* as shown in Fig. 20. Special measures should be taken when the fusion faces form angles greater than 135° or less than 45°. The effective length of a run of weld should be taken as the overall length less one leg length for each end that does not continue round a corner. The strength of the weld should be based on Table 25.

Table 25 Design strength, p_w, of fillet welds

grade of steel	electrode strength to BS 639[24] E43 N/mm²	E51 N/mm²	E51* N/mm²	other types N/mm²
40 or 43	215	215	—	
WR50 and 50	215	255	—	
55	—	255	275*	$0{\cdot}5U_e$ but $< 0{\cdot}54U_s$

* Applies only to electrodes having a minimum tensile strength of 550 N/mm² and a minimum yield strength of 450 N/mm².
U_e is the minimum tensile strength of the electrode based on all weld tensile tests as specified in BS 709[25].
U_s is the specified minimum ultimate tensile strength of the steel.



20 Fillet weld: effective throat thickness

Where welds are symmetrically disposed and are subject solely to direct tension or compression only, the strength of the welds may be taken as the design strength, p_y of the parent metal.

Where the weld is subject to a combination of forces and moments, the vectorial sum of the design stresses should be calculated, and it should be limited to the values given in Table 25.

Intermittent welds should not be used in connections subject to fatigue or in locations vulnerable to moisture penetration and corrosion.

14.3.2 Butt welds

Throat thickness for full penetration welds should be taken as the thickness of member. For partial penetration welds, it should be taken as the minimum depth of weld penetration, except that it should be taken as the actual depth less 3mm for V- or bevel welds. The depth should not be less than $2\sqrt{t}$, where t is the thickness in mm of the thinner connected part.

The design strength should be taken as that of the parent metal, provided that the weld is made with a suitable electrode.

Any eccentricity of partial penetration butt welds should be taken into account in calculating the stresses.

The strengths of the various types of welds are tabulated in the blue book.

15 Typical connections

In this *Manual* a number of typical connections for braced multistorey buildings of simple construction and for single-storey buildings, including portal frames, are described. For each connection, a procedure is listed that will enable bolts, welds and plates to be designed and bearing parts of the main members to be checked for the appropriate strengths.

The sequence and method of erection should be considered for safety and ease of erection when the connections are being designed.

For more detailed methods of design, and for 'standard' details reference should be made to:

Detailed design rules for steelwork connections[26] (SCI)

Manual on connections, Volume 1[27] (BCSA)

15.1 Column bases

15.1.1 General

Column bases should be of sufficient size, stiffness and strength to transmit safely the forces in the columns to the foundations. Linear pressure distribution may be assumed in the calculation of contact pressures. The maximum pressure on concrete foundations for factored loads should be limited to $0.4 f_{cu}$, where f_{cu} is the 28-day cube strength of the concrete.

15.1.2 Design of base plates

Base plates transmitting concentric loads may be designed by the empirical method using the following formulae:

- For I-, H-, channel, box and RHS sections, the minimum thickness should be not less than:

$$t = \left(\frac{2\cdot 5}{p_{yp}} w(a^2 - 0\cdot 3b^2)\right)^{1/2}$$

- In no case should it be less than the thickness of the column flanges
- For solid or hollow round columns the minimum should be not less than:

$$t = \left(\frac{w}{2\cdot 4p_{yp}} D_p(D_p - 0\cdot 9D)\right)^{1/2}$$

where a is the greater projection of the plate ignoring any oversizing beyond the column
b is the lesser projection of the plate ignoring any oversizing beyond the column
w is the pressure on the underside of the plate assuming uniform distribution
p_{yp} is the design strength of the plate $<270 N/mm^2$
D_p is the length of the side or diameter of the plate not less than $1.5\ (D + 75)$ mm
D is the diameter of the column

- If the pressure distribution is not uniform as when moments are transferred or if the base plate is not rectangular, the maximum bending moments in the base plate should be limited to $1\cdot 2\ p_{yp}Z$,

where p_{yp} is the design strength of the base plate, $< 270\ N/m^2$, and
Z is the elastic modulus of the base plate.

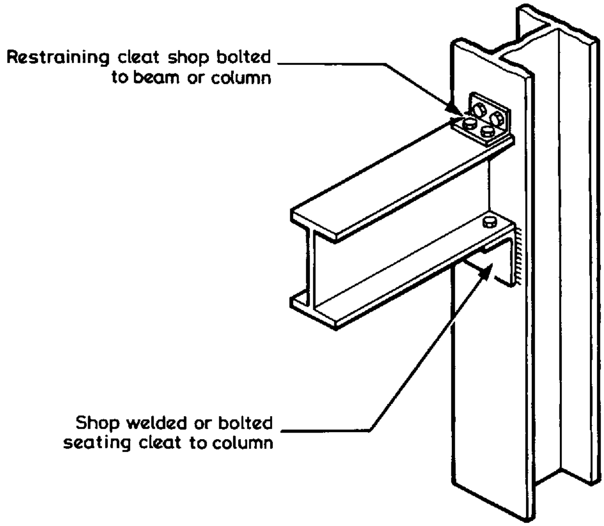
15.1.3 Design of gusset

The bending moments in any stiffening gusset should be limited to p_{yg} Z,
where p_{yg} is the design strength of the gusset < 270 N/mm^2, and
Z is the appropriate elastic modulus of the gusset.

15.2 Beam-to-column and beam-to-beam connections for simple construction

The different types of beam to column connections are shown below, together with the design procedure to be followed for each type.

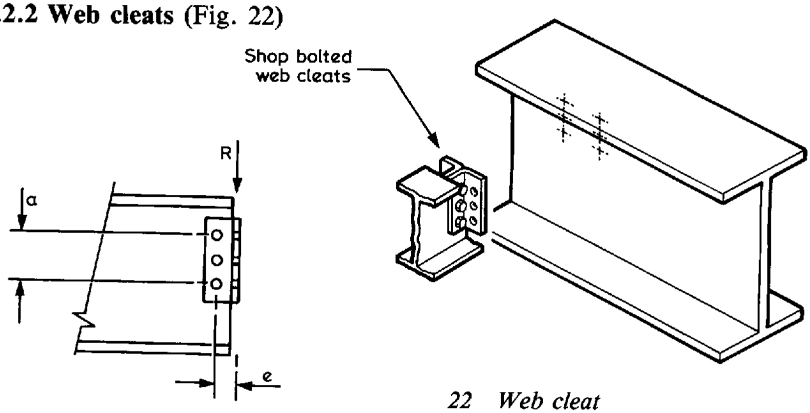
15.2.1 Top and bottom cleats (Fig. 21)



21 Top and bottom cleats

(a) Choose size of seating cleat angles
(b) Calculate the number of bolts required in shear and bearing on the lower cleat, which is assumed to support the whole of the vertical loading
(c) Alternatively, calculate the weld size to suit maximum length available
(d) Check buckling strength of beam web
(e) Check bearing strength at the root of the beam web
(f) Check bearing strength of angle cleat (area of bearing × design strength)
(g) Check bearing strength of column due to bolt loads where appropriate.

15.2.2 Web cleats (Fig. 22)



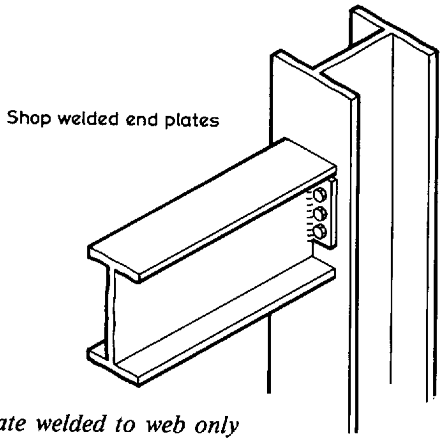
22 Web cleat

- *(a)* Choose cleat size, and calculate number and type of bolts
- *(b)* Calculate force in the outermost bolts connecting the cleats to the beam web, from shear and eccentricity
- *(c)* Check bolt strength in double shear on beam
- *(d)* Check bearing stress on the beam web and cleats
- *(e)* Calculate force in bolts connecting beam to column or beam
- *(f)* Check bolts in single shear on column or beam
- *(g)* Check bearing stress on cleats and column or beam
- *(h)* Check shear stress in cleats
- *(i)* Check reduced beam section for shear and moment if there is a notch.

Note 1. Where maximum edge distances cannot be achieved the bolt strengths should be reduced proportionally.

Note 2. Shear on bolt $= \sqrt{(F_s^2 + F_b^2)}$,
where F_s = reaction $R/2$ and $F_b = R \times e/a$;
e and a are shown on Fig. 22.

15.2.3 Thin flexible end plates welded only to web (Fig. 23)

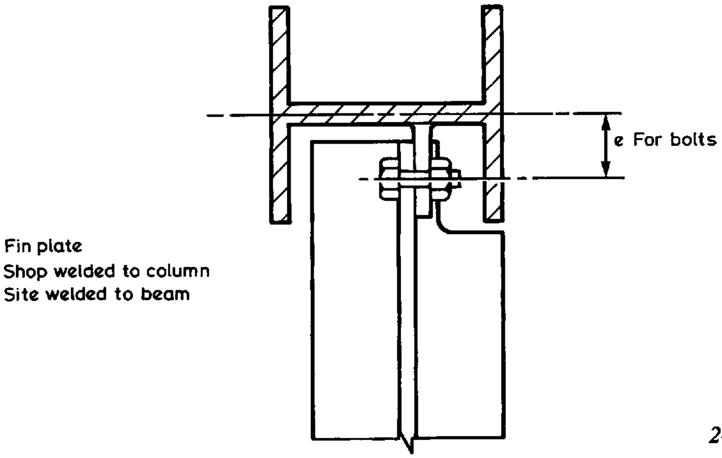


23 Thin flexible end plate welded to web only

(a) Choose plate size and number of bolts
(b) Calculate force in bolts
(c) Check bolt strength in single shear
(d) Check bearing stress on plate and column or beam
(e) Check shear stress in plate across net area after deducting hole areas
(f) Check shear in beam web over the depth of the end plate
(g) Choose fillet weld size to suit double length of weld (deducting amount equal to fillet size at each end of the run)
(h) Check reduced beam section for shear if there is a notch.

Note. Where maximum edge distances cannot be achieved the bolt strengths should be reduced proportionally.

15.2.4 Fin plates (Fig. 24)



24 Fin plate

(a) Choose fin plate size, number and grade 8.8 bolts
(b) Calculate force in outermost bolts from reaction and eccentricity moment
(c) Check bolt strength in single shear (reduce permissible shear values by 20%)
(d) Check bearing stress in web and the fin plate
(e) Check the shear stress in the plate across the net area after deducting hole areas
(f) Check bending of fin plate
(g) Check size of weld in shear and bending and choose a fillet weld size to suit double length of weld (reduce permissible shear values by 20%)
(h) Check reduced beam section for shear and bending if there is a notch.

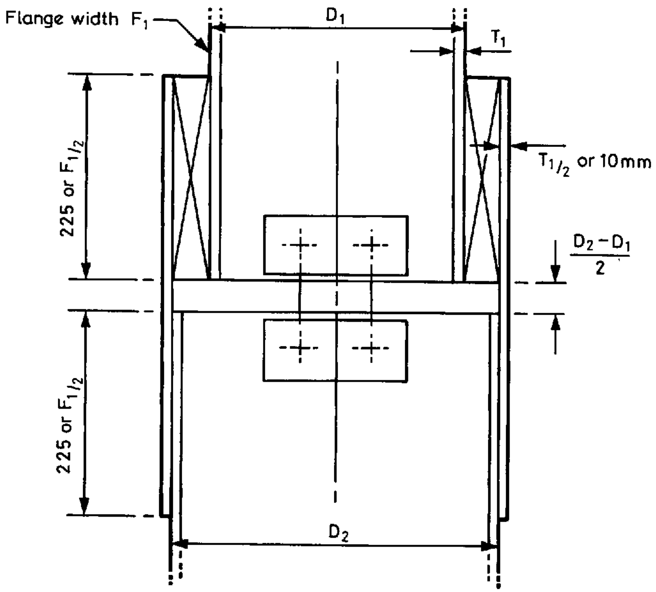
Note 1. Where maximum edge distances cannot be achieved the bolt strengths should be reduced proportionally.
2. Limit of beam depth to be 610 UB.
3. One vertical line of bolts only.
4. t_p or $t_w \leqslant 0 \cdot 5\ d_b$ for grade 43
$\leqslant 0 \cdot 5\ d_b$ for grade 50,
where d_b = bolt diameter.

15.3 Column-to-column splices

Column splices located should be adjacent to the floor and designed to meet the following requirements:

(a) they should hold the connected members in place
(b) the centroidal axis of the splice should coincide with the centroidal axis of the connected members
(c) they should provide continuity of stiffness about both axes and should also resist any tension.
(d) they should provide the tensile forces to comply with the robustness requirements of Section 7.

15.3.1 Column splices (ends prepared for contact in bearing) (Fig. 25)



25 Column splice — ends prepared for contact in bearing

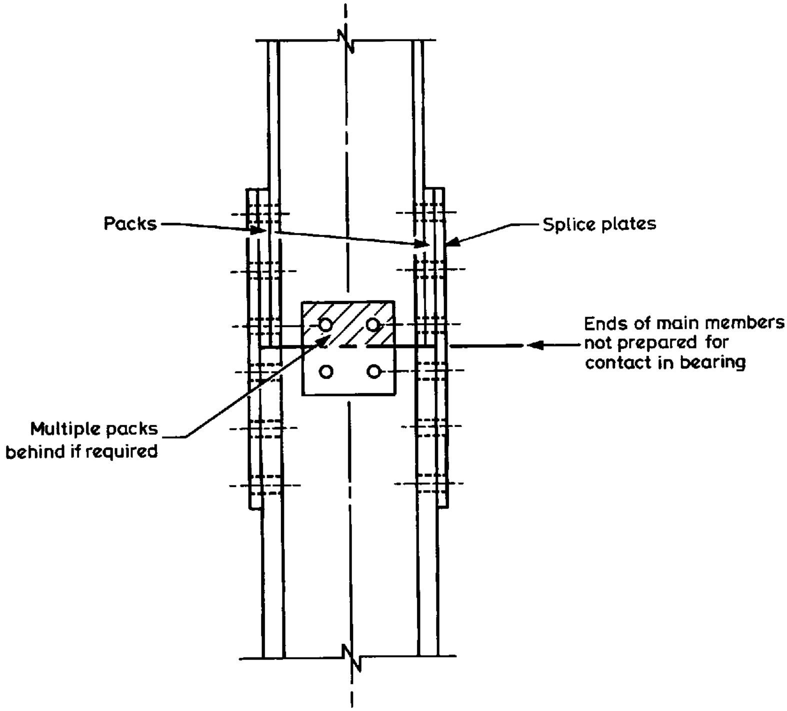
Splices should be designed for full contact bearing to resist the vertical loads. In addition, the following recommendations should be followed:

(a) the projection of the flange cover plates beyond the ends of the column members should be equal to the width of the flange of the upper column or 225 mm, whichever is greater
(b) the thickness of the flange cover plates should be half the thickness of the flange of the upper column or 10 mm, whichever is greater
(c) when the column sections to be joined are of the same size, then nominal web cover plates may be used
(d) when the column sections to be joined are of different sizes, then web cleats and a division plate should be used to give a load dispersal of 45°.

It should be noted that these splices may have to be designed to resist moments and/or tension because of robustness considerations. In these cases, the permissible shear value of the bolts may have to be reduced, and the tensile capacities of the division and flange plates should be checked.

15.3.2 Column-to-column splices (ends not prepared for contact in bearing) (Fig. 26)

All the forces and moments should be wholly transmitted through the bolts and splice plates, and not by bearing between column members.

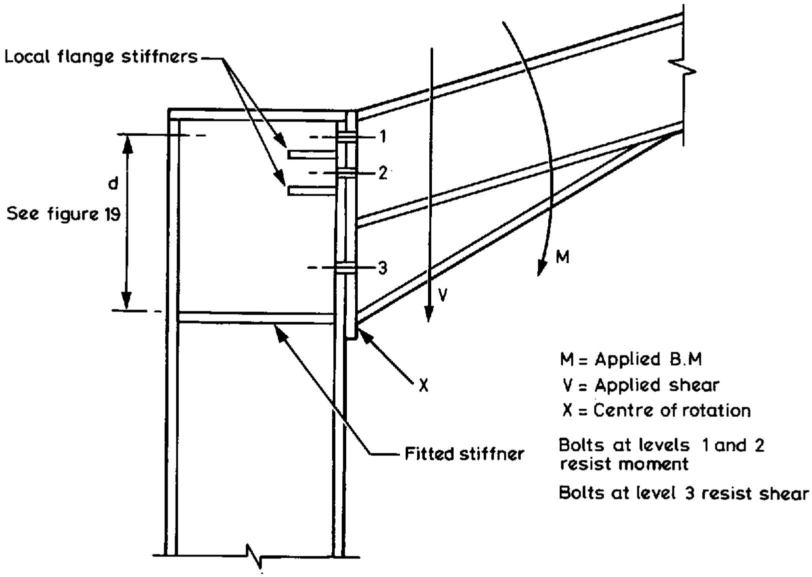


26 Column splice — ends not prepared for contact in bearing

Notes
1. The axial load is carried by the web and flanges.
2. The bending moments are deemed to be carried by the flanges.
(a) choose the splice plate size, number and type of bolts
(b) calculate the forces in the bolts arising from the axial load and moment, if any
(c) check the bolt strength in single shear
(d) check the bearing stress in flanges and in splice plate
(e) check the tensile capacity and shear stress in the plate across the net area deducting hole areas.

15.4 Portal frame connections

15.4.1 Portal frame — haunch (Fig. 27)



27 Portal frame haunch

(a) Assume the number and type of bolts required at 1 and 2 (see Fig. 27) to resist the factored bending moment, and locate them to obtain the maximum lever arm.

(b) Using the force distribution shown in Fig. 19, calculate the resistance moment. If this is less than the applied moment increase the number and/or size of bolts.

(c) Check the thickness of the end plate required to resist the bending moments caused by the bolt tension. Double-curvature bending of the plates may be assumed since bolts occur on both sides of the web.

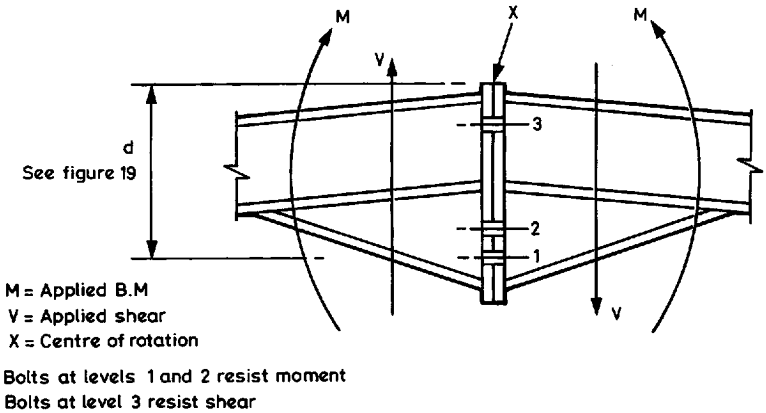
(d) Check that the sum of the horizontal bolt forces at 1 and 2 (see Fig. 27) can be resisted in shear by the stanchion.

(e) Check the bending stresses in the stanchion flange caused by the bolt tension; normally local stiffener will need to be added.

(f) Calculate the additional bolts required at 3 (see Fig. 27) to resist the applied vertical shear by checking the single shear capacity of the bolts and the bearing on the end plate and on the stanchion flange.

(g) Check for reversal of moment due to wind, as this may govern the design of bolts (3 on Fig. 27) and of the stiffners.

15.4.2 Portal frame — ridge (Fig. 28)



28 Portal frame ridge

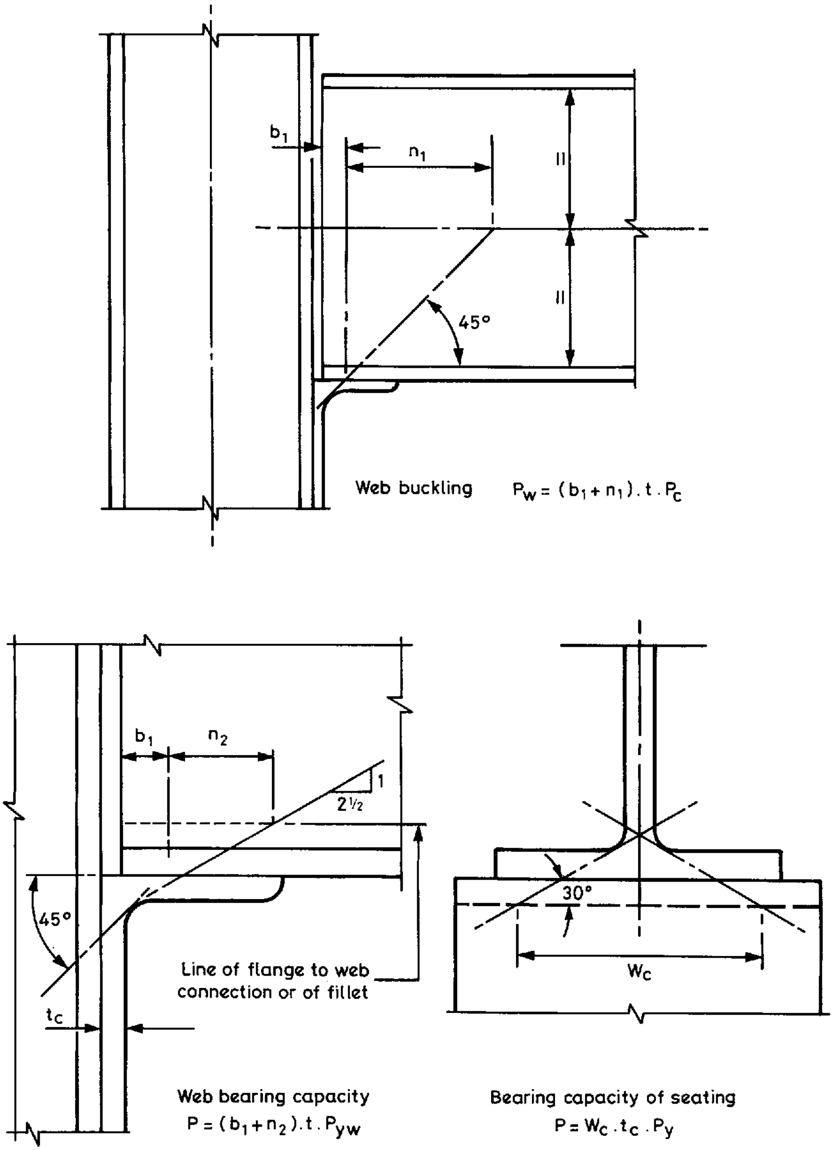
(a) Assume the number and type of bolts required at 1 and 2 (see Fig. 28) to resist the bending moment and locate them to obtain maximum lever arm.
(b) Using the distribution of force shown in Fig. 19, calculate the resisting moment. If it is less than the applied moment increase the number or size of bolts.
(c) Check the thickness of end plates to resist bending moments caused by the bolt tension; provide local stiffening for the end plates as necessary.
(d) Calculate the bolts required to resist the applied horizontal shear at 3 (see Fig. 28) by checking their single shear capacity and the bearing capacities on the end plates.

15.5 Web buckling and bearing

This check should be carried out when heavy loads (or reactions) are applied to unstiffened webs, e.g. it applies to beams supported on the bottom flange with the load applied to the top flange; to a column supported by a beam; to a beam continuous over a column; and to web resisting compression forces from haunches in portals.

Web buckling and bearing may be checked as described below, the dimensions being shown in Fig. 29.

where b_1 is the length of stiff bearing
n_1 is as shown on Fig. 29
t is the web thickness
p_c is obtained from Table D3 using $\lambda = 2.5\ d/t$ (where d is the depth of web); provided that the flange through which the load or reaction is applied is effectively restrained against rotation relative to the web and lateral movement relative to the other flange; if these conditions are not met, BS 5950 should be consulted.
P_w is the web buckling resistance of the unstiffened web
P_{yw} is the design strength of the web
n_2 is as shown in Fig. 29
t_c is the thickness of the cleat
W_c is as shown in Fig. 29.



29 Unstiffened web and bearings

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Appendix A Moment capacities M_{cx} for fully restrained beams, critical values of L_E for maximum M_{cx}, buckling resistance moments M_B for beams with intermediate restraints and I for UB sections

Table A1 Grade 43 steel

serial size	mass, kg/m	I,cm^4 x-x axis	maxM_{cx} fully restrained, kN-m	critial values ofL_{Em}	buckling resistance moments for effective lengths between restraints of $L_E m =$ 4·0	5·0	6·0	7·0	8·0	9·0	10·0
914×419	388	719 000	4690	3·89	4659	4330	4000	3690	3380	3100	2850
	343	625 000	4110	3·82	4060	3760	3460	3170	2890	2640	2410
914×305	289	505 000	3340	2·67	2880	2540	2220	1940	1710	1510	1360
	253	437 000	2890	2.63	2460	2160	1870	1620	1410	1240	1100
	224	376 000	2520	2·57	2120	1840	1580	1360	1170	1020	904
	201	326 000	2220	2·50	1830	1580	1340	1140	978	847	743
838×292	226	340 000	2430	2·56	2040	1780	1540	1330	1160	1020	908
	194	279 000	2030	2·48	1670	1440	1230	1050	905	788	694
	176	246 000	1800	2·43	1470	1250	1060	899	769	665	583
762×267	197	240 000	1900	2.40	1520	1310	1120	961	836	736	656
	173	205 000	1640	2·28	1290	1100	927	789	679	593	524
	147	169 000	1370	2·22	1050	882	735	617	526	454	398
686×254	170	170 000	1490	2·26	1170	1000	854	735	640	564	504
	152	150 000	1320	2·22	1030	870	735	627	541	474	421
	140	136 000	1210	2.19	925	778	653	553	474	413	365
	125	118 000	1060	2·14	796	662	550	461	392	339	298
610×305	238	208 000	1980	2·97	1800	1630	1480	1340	1220	1110	1020
	179	152 000	1460	2·87	1300	1170	1040	922	822	736	665
	149	125 000	1210	2·81	1070	947	834	732	644	571	511
610×229	140	112 000	1100	2·05	815	685	580	497	432	382	342
	125	98 600	975	2·02	709	591	494	420	362	318	283
	113	87 400	872	1·98	623	513	425	358	307	267	237
	101	75 700	792	1·91	546	443	361	300	255	220	194
533×210	122	76 200	848	1·91	603	506	428	369	323	287	258
	109	66 700	747	1·87	518	429	359	306	266	235	210
	101	61 700	694	1·85	475	390	324	275	237	209	186
	92	55 400	652	1·80	432	350	288	241	207	181	160
	82	47 500	566	1·75	364	291	236	196	166	144	127

Table A2 Grade 43 steel

serial size	mass, kg/m	I,cm^4 x-x axis	maxM_{cx} fully restrained, kN-m	Critical values of L_{Em}	buckling resistance moments for effective lengths between restraints of $L_E m =$							
					2·0	2·5	3	3·5	4	4·5	5	6
457×191	98	45 700	591	1·77	571	525	467	442	400	365	334	283
	89	41 000	533	1·74	511	469	429	391	352	323	291	244
	82	37 100	503	1·68	478	437	396	357	321	289	261	217
	74	33 400	456	1·66	431	393	355	319	285	255	230	189
	67	29 400	404	1·64	380	345	310	277	246	219	195	159
457×152	60	25 500	352	1·29	298	260	224	193	168	147	130	105
	52	21 300	300	1·25	249	215	183	156	134	116	102	81
406×178	74	27 300	412	1·61	385	351	317	286	257	232	210	175
	67	24 300	371	1·59	345	313	282	252	226	202	182	151
	60	21 500	327	1·57	303	274	245	218	194	173	154	126
	54	18 600	289	1·53	264	238	211	186	164	145	129	104
406×140	46	15 600	244	1·20	199	171	145	124	107	93	82	66
	39	12 500	198	1·16	157	133	112	93	79	68	60	47
356×171	67	19 500	333	1·59	310	283	156	232	210	190	173	147
	57	16 100	278	1·55	256	232	208	186	167	150	135	112
	51	14 200	246	1·53	225	203	182	161	143	128	114	94
	45	12 100	213	1·50	193	173	154	135	119	105	93	76
356×127	39	10 100	180	1·07	137	115	96	82	70	61	54	44
	33	8 200	148	1·04	109	90	75	62	53	46	40	32
305×165	54	11 700	232	1·57	215	196	178	161	146	132	121	1102
	46	9 950	199	1·54	183	166	149	133	119	107	97	81
	40	8 520	172	1·51	157	141	126	112	99	88	79	65
305×127	48	9 500	194	1·12	155	135	118	103	91	82	74	63
	42	8 140	168	1·09	131	112	96	84	74	65	59	49
	37	7 160	148	1·07	114	96	82	71	62	54	49	40
305×102	33	6 490	132	0·87	86	70	58	49	42	37	33	27
	28	5 420	112	0·84	70	56	45	38	33	28	25	20
	25	4 390	92	0·80	54	43	34	28	24	21	18	
254×146	43	6 560	156	1·41	129	126	113	102	92	84	77	65
	37	5 560	133	1·38	118	105	93	83	74	67	61	51
	31	4 440	109	1·33	94	83	73	84	56	50	45	37
254×102	28	4 010	97	0·89	66	54	46	39	34	30	27	23
	25	3 410	84	0·86	54	44	37	31	27	24	21	17
	22	2 870	72	0·83	45	36	29	24	21	18	16	13
203×133	30	2 890	86	1·29	73	65	58	52	47	42	39	33
	25	2 360	71	1·25	60	53	46	41	36	32	29	24

Table A3 Grade 50 steel

serial size	mass, kg/m	I, cm^4 x-x axis	max M_{cx} fully restrained, kN-m	critial values of L_{Em}	buckling resistance moments for effective lengths between restraints of $L_E m =$ 4·0	5·0	6·0	7·0	8·0	9·0	10·0
914×419	388	719 000	6020	3·41	5760	5320	4870	4430	4000	3620	3280
	343	625 000	5270	3·36	5020	4620	4210	3800	3410	3060	2750
914×305	289	505 000	4280	2·35	3510	3030	2590	2220	1920	1680	1490
	253	437 000	3710	2·31	3000	2570	2170	1840	1580	1370	1210
	224	376 000	3240	2·26	2580	2190	1830	1540	1310	1130	984
	201	326 000	2840	2·21	2220	1860	1540	1280	1080	927	805
838×292	226	340 000	3110	2·25	2480	2110	1780	1510	1290	1130	991
	194	279 000	2600	1·19	2030	1700	1420	1180	1000	863	753
	176	246 000	2320	2·14	1770	1470	1220	1010	848	726	630
762×267	197	240 000	2440	2·05	1830	1530	1280	1080	925	806	712
	173	205 000	2110	2·01	1550	1280	1060	880	747	646	567
	147	169 000	1760	1·95	1290	1020	831	685	575	492	428
686×254	170	170 000	1910	1⅓8	1410	1170	975	823	706	617	547
	152	150 000	1700	1·96	1230	1010	835	698	595	516	455
	140	136 000	1550	1·83	1110	903	739	614	520	449	393
	125	118 000	1360	1·89	948	765	619	510	428	367	320
610×305	238	208 000	2540	2·60	2210	1980	1760	1570	1400	1270	1150
	179	152 000	1880	2·51	1600	1410	1230	1070	935	826	738
	149	125 000	1550	2·47	1300	1140	979	841	728	636	563
610×229	140	112 000	1410	1·80	968	792	654	551	474	415	369
	125	98 600	1250	1·77	840	679	555	464	396	344	304
	113	87 400	1120	1·75	735	588	475	394	333	288	253
	101	75 700	1020	1·67	643	504	402	329	276	236	207
533×210	122	76 200	1090	1·68	712	580	481	480	353	311	278
	109	66 700	959	1·65	609	489	401	337	289	253	255
	101	61 700	891	1·63	557	444	361	301	258	225	199
	92	55 400	841	1·58	506	397	319	264	224	194	171
	82	47 000	731	1·54	424	328	260	213	179	154	135

Table A4 Grade 50 steel

serial size	mass, kg/m	I,cm^4 x-x axis	maxM_{cx} fully restrained, kN-m	Critical values of L_{Em}	buckling resistance moments for effective lengths between restraints of $L_E m=$							
					2·0	2·5	3	3·5	4	4·5	5	6
457×191	98	45 700	758	1·55	705	647	584	522	468	424	380	316
	89	41 000	683	1·53	633	567	520	496	410	368	329	271
	82	37 100	650	1·48	593	536	478	423	374	331	295	240
	74	33 400	589	1·46	535	482	428	377	330	291	258	208
	67	29 400	452	1·44	470	422	312	325	283	248	219	174
457×152	60	25 500	454	1·13	364	310	261	220	187	162	142	113
	52	21 300	387	1·10	303	255	212	176	149	128	111	87
406×178	74	27 300	532	1·40	477	429	381	337	297	264	236	194
	67	24 300	479	1·39	427	382	338	296	260	229	204	166
	60	21 500	422	1·38	374	334	293	255	222	195	172	139
	54	18 600	373	1·34	327	289	251	217	187	163	143	114
406×140	46	15 600	315	1·06	241	202	167	140	118	102	89	71
	39	12 500	256	1·02	190	156	127	105	87	74	64	50
356×171	67	19 500	430	1·39	384	346	308	274	243	217	196	162
	57	16 100	359	1·36	317	283	249	218	192	170	151	123
	51	14 200	318	1·34	279	247	216	188	164	144	127	103
	45	12 100	244	1·32	238	210	182	157	136	118	104	82
356×127	39	10 100	232	0·94	164	134	110	91	77	67	59	47
	33	8 200	192	0·91	131	105	84	69	58	49	43	34
305×165	54	11 700	300	1·37	267	240	214	190	169	151	136	113
	46	9 950	257	1·35	226	202	178	156	138	122	109	89
	40	8 520	222	1·32	193	171	150	130	114	99	88	71
305×127	48	9 500	251	0·97	187	159	135	117	103	91	82	69
	42	8 140	217	0·95	158	132	111	94	82	72	64	53
	37	7 160	192	0·94	137	113	94	79	68	59	53	43
305×102	33	6 490	170	0·76	101	79	64	54	46	40	36	29
	28	5 420	144	0·74	81	63	50	41	35	30	27	22
	25	4 390	120	0·70	63	48	37	30	26	22	19	
254×146	43	6 560	202	1·23	172	153	135	119	106	95	86	72
	37	5 560	172	1·20	145	127	111	96	85	75	67	56
	31	4 440	125	1·17	115	99	85	73	63	56	49	40
254×102	28	4 010	127	0·78	77	62	51	43	37	33	29	24
	25	3 410	109	0·76	64	50	41	34	29	26	23	19
	22	2 870	93	0·73	52	40	32	27	23	20	17	14
203×133	30	2 890	111	1·12	90	79	69	60	53	48	43	36
	25	2 360	82	1·09	73	63	54	46	40	36	32	26

Appendix B Bending strength, p_b, tables

Table B1 Bending strength, p_b, (in N/mm²) for rolled sections with equal flanges for p_y = 265 N/mm²

λ \ x	5	10	15	20	25	30	35	40	45	50
30	265	265	265	265	265	265	265	265	265	265
35	265	265	265	265	265	265	265	265	265	265
40	265	265	265	265	265	264	264	264	263	263
45	265	265	261	258	256	255	254	254	254	254
50	265	261	253	249	247	246	245	244	244	244
55	265	255	246	241	238	236	235	235	234	234
60	265	250	239	233	229	227	226	225	224	224
65	265	245	232	225	221	218	216	215	214	214
70	265	240	225	217	212	209	207	205	204	204
75	263	235	219	210	204	200	198	196	195	194
80	260	230	213	202	196	191	189	187	185	184
85	257	226	207	195	188	183	180	178	176	175
90	254	222	201	188	180	175	171	169	167	166
95	252	217	196	182	173	167	163	160	158	157
100	249	213	190	176	166	160	156	153	150	149
105	247	209	185	170	160	153	148	145	143	141
110	244	206	180	164	154	147	142	138	136	134
115	242	202	176	159	148	140	135	132	129	127
120	240	198	171	154	142	135	129	125	123	121
125	237	195	167	149	137	129	124	120	117	115
130	235	191	163	144	132	124	119	114	111	109
135	233	188	159	140	128	119	114	109	106	104
140	231	185	155	136	124	115	109	105	102	99
145	229	182	152	132	120	111	105	101	97	95
150	227	179	148	129	116	107	101	97	93	91
155	225	175	145	125	112	103	97	93	89	87
160	223	173	142	122	109	100	94	89	86	83
165	221	170	139	119	106	97	91	86	83	80
170	219	167	136	116	103	94	88	83	80	77
175	217	165	133	113	100	91	85	80	77	74
180	215	162	130	110	97	88	82	77	74	71
185	213	160	128	108	95	86	79	75	71	69
190	211	157	125	105	92	83	77	73	69	66
195	209	155	123	103	90	81	75	70	67	64
200	207	153	120	101	88	79	73	68	65	62
210	204	148	116	96	84	75	69	64	61	58
220	200	144	112	93	80	71	65	61	58	55
230	197	140	108	89	77	68	62	58	54	52
240	194	136	104	86	74	65	59	55	52	49
250	190	132	101	83	71	63	57	52	49	47
λ_L	70·7	46·7	42·3	40·7	40·0	39·7	39·4	39·3	39·2	39·1

Table B2 Bending strength, p_b, (in N/mm²) for rolled sections with equal flanges for p_y = 275 N/mm²

λ \ x	5	10	15	20	25	30	35	40	45	50
30	275	275	275	275	275	275	275	275	275	275
35	275	275	275	275	275	275	275	275	275	275
40	275	275	275	275	274	273	272	272	272	272
45	275	275	269	266	264	263	263	263	263	263
50	275	269	261	257	255	253	253	252	252	251
55	275	262	254	248	246	244	243	242	241	241
60	275	258	246	240	236	234	233	232	231	230
65	275	252	239	232	227	224	223	221	221	220
70	274	247	232	223	218	215	213	211	210	209
75	271	242	225	215	209	206	203	201	200	199
80	269	237	219	208	201	196	193	191	190	189
85	265	233	213	200	193	188	184	182	180	179
90	262	228	207	193	185	179	175	173	171	169
95	260	224	201	186	177	171	167	164	162	160
100	257	219	195	180	170	164	159	156	153	152
105	254	215	190	174	163	156	151	148	146	144
110	252	211	185	168	157	150	144	141	138	136
115	250	207	180	162	151	143	138	134	131	129
120	247	204	175	157	145	137	132	128	125	123
125	245	200	171	152	140	132	126	122	119	116
130	242	196	167	147	135	126	120	116	113	111
135	240	193	162	143	130	121	115	111	108	106
140	238	190	159	139	126	117	111	106	103	101
145	236	186	155	135	122	113	106	102	99	96
150	233	183	151	131	118	109	102	98	95	92
155	231	180	148	127	114	105	99	94	91	88
160	229	177	144	124	111	101	95	90	87	84
165	227	174	141	121	107	98	92	87	84	81
170	225	171	138	118	104	95	89	84	81	78
175	223	169	135	115	101	92	86	81	78	75
180	221	166	133	112	99	89	83	78	75	72
185	219	163	130	109	96	87	80	76	72	70
190	217	161	127	107	93	84	78	73	70	67
195	215	158	125	104	91	82	76	71	68	65
200	213	156	122	102	89	80	74	69	65	63
210	209	151	118	98	85	76	70	65	62	59
220	206	147	114	94	81	72	66	62	58	55
230	202	143	110	90	78	69	63	58	55	52
240	199	139	106	87	74	66	60	56	52	50
250	195	135	103	84	72	63	57	53	50	47
λ_L	68·4	45·5	41·3	39·9	39·2	38·9	38·7	38·6	38·5	38·4

Table B3 Bending strength, p_b, (in N/mm²) for rolled sections with equal flanges for p_y = 340 N/mm²

λ \ x	5	10	15	20	25	30	35	40	45	50
30	340	340	340	340	340	340	340	340	340	340
35	340	340	340	340	340	340	339	339	339	339
40	340	333	333	320	328	327	327	326	326	326
45	340	333	323	318	316	315	314	314	313	313
50	340	322	312	307	304	302	301	301	300	300
55	340	315	303	296	292	290	288	287	286	286
60	337	308	293	285	280	277	275	274	273	272
65	333	301	283	273	268	264	262	260	259	258
70	329	294	274	263	256	251	248	246	245	244
75	325	287	265	252	244	239	235	233	231	230
80	321	281	257	242	232	227	223	220	218	216
85	318	275	248	232	222	215	211	207	205	203
90	214	269	240	223	211	204	199	196	193	191
95	311	263	232	213	201	194	188	185	182	180
100	307	257	225	205	192	184	178	174	171	169
105	304	252	218	197	184	175	169	165	161	159
110	301	246	211	189	176	166	160	156	152	150
115	297	241	205	182	168	159	152	147	144	142
120	294	236	199	176	161	151	145	140	136	134
125	291	231	193	179	155	145	138	133	129	127
130	288	227	188	164	148	138	131	126	123	120
135	285	222	183	158	143	133	125	120	117	114
140	282	218	178	153	138	127	120	115	111	108
145	270	213	173	148	133	122	115	110	106	103
150	276	209	168	144	128	118	110	105	101	99
155	273	205	164	139	124	113	106	101	97	94
160	270	207	160	135	120	109	102	97	93	90
165	267	197	156	132	116	106	98	93	89	86
170	265	194	153	128	112	102	95	90	86	83
175	262	190	149	125	109	99	92	86	82	79
180	259	187	146	121	106	96	88	83	79	76
185	257	184	142	118	103	93	86	80	77	74
190	254	180	139	115	100	90	83	78	74	71
195	215	177	136	113	98	87	80	75	71	68
200	249	174	134	110	95	85	78	73	69	66
210	244	168	128	105	90	81	74	69	65	62
220	239	163	123	101	86	77	70	65	61	58
230	234	158	119	96	82	73	66	61	58	55
240	230	153	115	93	79	70	63	58	55	52
250	225	149	111	89	76	67	60	56	52	49
λ_L	56·5	39·6	36·6	35·1	34·1	34·9	34·7	34·6	34·5	34·4

Table B4 Bending strength, p_b, (in N/mm^2) for rolled sections with equal flanges for p_y = 355 N/mm^2

λ \ x	5	10	15	20	25	30	35	40	45	50
30	340	355	355	355	355	355	355	355	355	355
35	355	355	355	354	353	353	352	352	352	352
40	355	352	346	342	341	340	339	339	339	338
45	355	344	335	320	328	327	326	325	325	325
50	355	335	324	318	315	313	312	311	311	311
55	354	327	314	306	302	300	298	297	297	296
60	350	319	303	294	289	286	284	283	282	281
65	336	312	293	283	276	273	270	268	267	266
70	341	305	283	271	264	259	256	254	252	251
75	337	298	274	260	251	246	242	240	238	236
80	333	291	265	249	239	233	229	226	224	222
85	329	284	256	238	228	221	216	213	210	209
90	326	278	247	228	217	209	204	200	198	196
95	322	271	239	219	206	198	193	189	186	184
100	318	265	231	210	197	188	182	178	175	173
105	315	260	224	202	188	178	172	168	165	162
110	311	254	217	194	179	170	163	159	155	153
115	308	248	210	186	171	162	155	150	147	144
120	305	243	204	180	166	154	147	142	139	136
125	301	238	198	173	157	147	140	135	131	129
130	298	233	192	167	151	141	133	128	125	122
135	295	228	187	161	145	135	127	122	118	116
140	292	223	181	156	140	129	122	117	113	110
145	288	219	176	151	135	124	117	111	108	105
150	285	214	172	146	130	119	112	107	103	100
155	282	210	167	142	126	115	107	102	98	95
160	279	206	163	138	121	111	103	98	94	91
165	276	202	159	134	118	107	100	94	90	87
170	273	198	155	130	114	103	96	91	87	84
175	270	195	152	126	111	100	93	87	83	80
180	268	191	148	123	107	97	89	84	80	77
185	256	188	145	120	104	94	87	81	77	74
190	262	184	142	117	101	91	84	79	75	72
195	259	181	139	114	99	88	81	76	72	69
200	257	178	136	111	96	86	79	74	70	67
210	251	172	130	106	91	81	74	69	65	62
220	246	166	125	102	87	77	70	65	62	59
230	241	161	121	98	83	74	67	62	58	55
240	236	156	116	94	80	70	64	59	55	52
250	231	151	112	90	77	67	61	56	52	50
λ_L	54·4	38·6	35·7	34·8	34·3	34·1	33·9	33·9	33·8	33·7

Appendix C Axial and bending capacities of UC columns (grade 50 steel)

The Table is derived from the following assumptions and should be used for the simple design method only:

- axial capacities $A_g p_c$ (kN are based on an effective length L_E of 0·85 × storey height
- bending capacity values M_{bs} (kN-m) are about the x-x plane and are based on an effective length of 0·5 × storey height
- bending capacity values $p_y Z_y$ (kN-m) are about the y-y plane, and are unrelated to height
- tabulated values give the capacities at the storey heights indicated
- interpolation may be used.

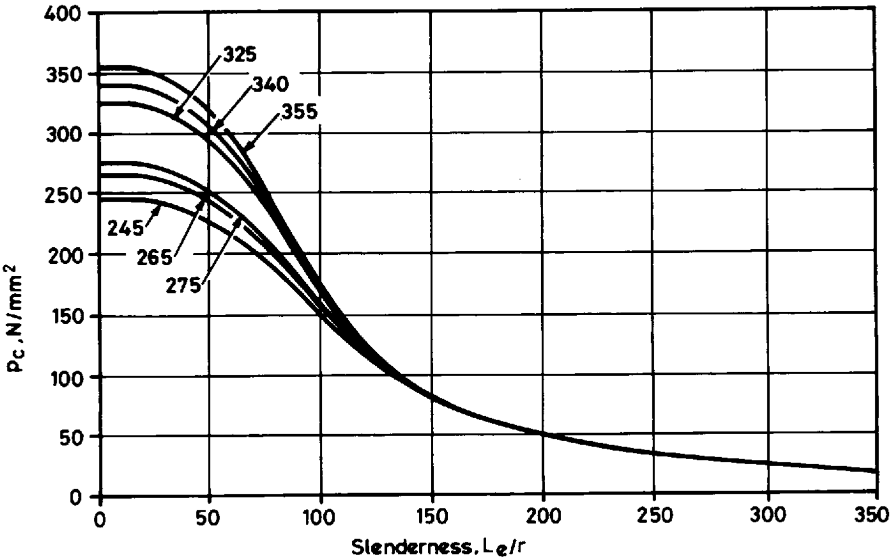
UC section — grade 50		storey height, m 0·0	3·0	4·0	5·0	6·0
356 × 406 × 340	$A_g p_c$	14700	13640	12650	11690	10660
$p_y Z_y$ = 789	M_{bs}	2380	2380	2380	2380	2380
356 × 406 × 287	$A_g p_c$	12400	11865	11130	10470	9740
$p_y Z_y$ = 660	M_{bs}	1980	1980	1980	1980	1980
356 × 406 × 235	$A_g p_c$	10200	9630	9110	8565	7950
$p_y Z_y$ = 534	M_{bs}	1590	1590	1590	1590	1590
356 × 368 × 202	$A_g p_c$	8770	8200	7690	7195	6610
$p_y Z_y$ = 428	M_{bs}	1350	1350	1350	1350	1350
356 × 368 × 177	$A_g p_c$	7680	7165	6740	6280	5765
$p_y Z_y$ = 374	M_{bs}	1180	1180	1180	1180	1170
356 × 368 × 153	$A_g p_c$	6630	6180	5800	5400	4955
$p_y Z_y$ = 321	M_{bs}	1010	1010	1010	1010	999
356 × 368 × 129	$A_g p_c$	5610	5198	4868	4538	4274
$p_y Z_y$ = 269	M_{bs}	843	843	843	843	843
305 × 305 × 283	$A_g p_c$	12200	10870	9970	8995	7980
$p_y Z_y$ = 520	M_{bs}	1730	1730	1730	1730	1650
305 × 305 × 240	$A_g p_c$	10400	9425	8715	7925	7070
$p_y Z_y$ = 432	M_{bs}	1450	1450	1450	1450	1370
305 × 305 × 198	$A_g p_c$	8570	7735	7110	6475	5750
$p_y Z_y$ = 350	M_{bs}	911	911	911	905	856
305 × 305 × 137	$A_g p_c$	5950	5350	4920	4420	3920
$p_y Z_y$ = 235	M_{bs}	782	782	782	775	737
305 × 305 × 118	$A_g p_c$	5100	4570	4175	3780	3330
$p_y Z_y$ = 200	M_{bs}	663	663	663	655	619
305 × 305 × 97	$A_g p_c$	4370	3885	3555	3190	2800
$p_y Z_y$ = 169	M_{bs}	564	564	564	553	521
254 × 254 × 167	$A_g p_c$	7210	6230	5580	4890	4160
$p_y Z_y$ = 252	M_{bs}	823	823	823	781	727

UC section — grade 50		storey height, m				
		0·0	3·0	4·0	5·0	6·0
254 × 254 × 132	$A_g p_c$	5750	4940	4425	3850	3265
$p_y Z_y$ = 196	M_{bs}	636	636	636	600	557
254 × 254 × 107	$A_g p_c$	4660	3990	3560	3075	2605
$p_y Z_y$ = 155	M_{bs}	507	507	507	476	441
254 × 254 × 89	$A_g p_c$	3880	3310	2855	2555	2155
$p_y Z_y$ = 129	M_{bs}	418	418	414	392	363
254 × 254 × 73	$A_g p_c$	3300	2795	2480	2120	1775
$p_y Z_y$ = 108	M_{bs}	351	351	349	325	301
203 × 203 × 86	$A_g p_c$	3740	2965	2490	2030	1610
$p_y Z_y$ = 102	M_{bs}	333	333	314	286	255
203 × 203 × 71	$A_g p_c$	3100	2450	2060	1670	1325
$p_y Z_y$ = 83	M_{bs}	273	273	256	233	208
203 × 203 × 60	$A_g p_c$	2690	2100	1745	1380	1095
$p_y Z_y$ = 70	M_{bs}	231	231	215	194	172
203 × 203 × 52	$A_g p_c$	2360	1830	1515	1215	945
$p_y Z_y$ = 61	M_{bs}	202	202	187	169	149
203 × 203 × 46	$A_g p_c$	2090	1615	1330	1055	825
$p_y Z_y$ = 53	M_{bs}	176	176	163	147	130
152 × 152 × 37	$A_g p_c$	1680	1085	800	580	430
$p_y Z_y$ = 32	M_{bs}	110	102	88	74	61
152 × 152 × 30	$A_g p_c$	1360	865	630	455	340
$p_y Z_y$ = 26	M_{bs}	87	80	70	58	47
152 × 152 × 23	$A_g p_c$	1060	650	465	335	245
$p_y Z_y$ = 18	M_{bs}	65	59	50	42	34

Appendix D Compressive strengths, p_c, for sections

Table D1 Compressive strength p_c for

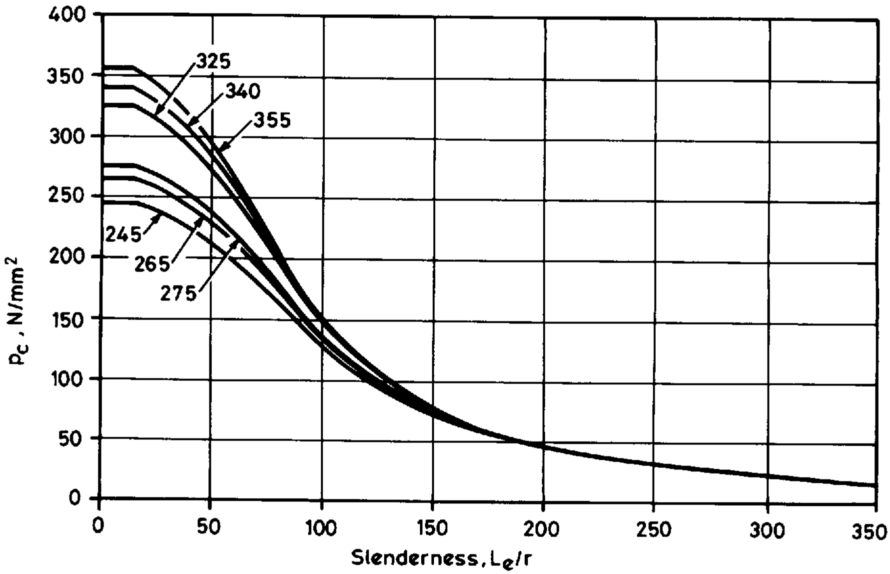
section	axis of buckling
hot-rolled hollow sections	major and minor
rolled I-sections	major
rolled I-sections with plates type A	major
rolled I- or H-sections with plates type B	minor



Slenderness λ

Table D2 Compressive strength p_c for

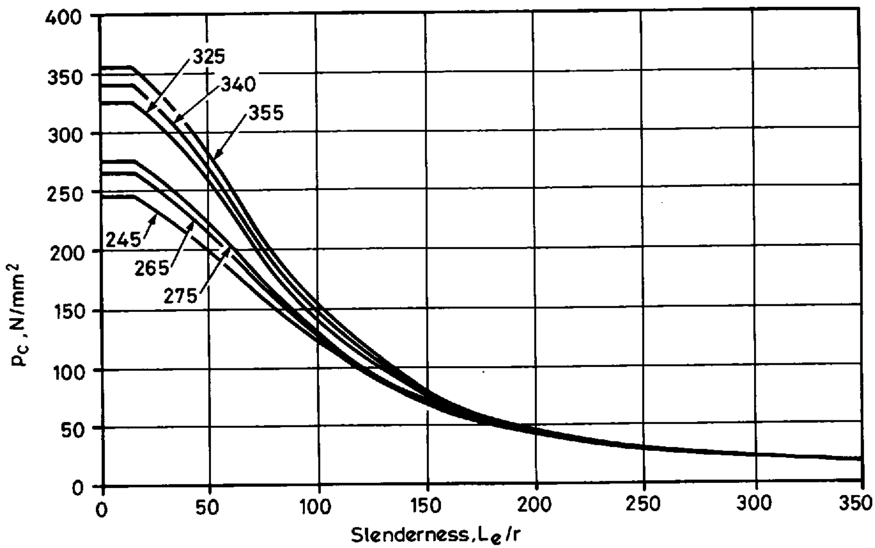
section	axis of buckling
rolled I-sections	minor
rolled H-sections < 40 mm flange	major
rolled, flat or square bars < 40 mm	major
rolled I- or H-sections with plates type B	
up to 40 mm	major
over 40 mm	minor



Slenderness λ

Table D3 Compressive strength p_c for

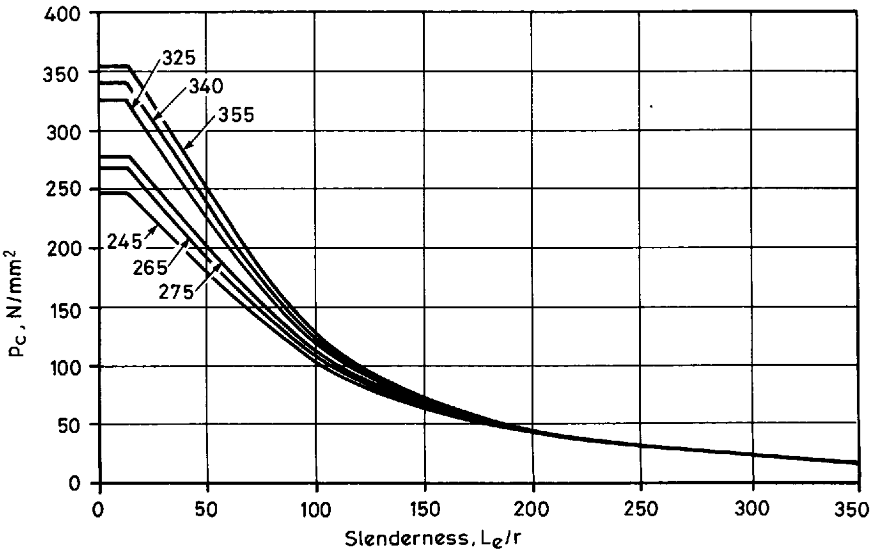
section	axis of buckling
rolled H-sections < 40 mm	minor
rolled H-sections > 40 mm	major
rolled I- or H-sections with plates type B > 40 mm	major
round, square or flat bars > 40 mm thick	major and minor
rolled angles, channels or T-sections, compound, laced or battened members	any axis



Slenderness λ

Table D4 Compressive strength p_c for

section	axis of buckling
rolled H-sections > 40 mm	minor



Slenderness λ

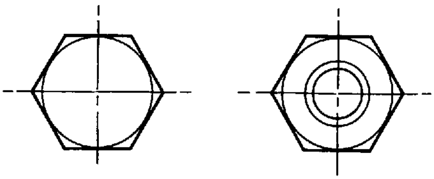
Appendix E Design data

contract	job no.
general description, intended use, unusual environment conditions	
site constraints	
stability provisions	
movement joints	
loading	
fire resistance	
durability	
soil conditions and foundation design	
performance criteria	
materials	
ground slab construction	
other data	

Appendix F Indentification marks for bolts, nuts and washers

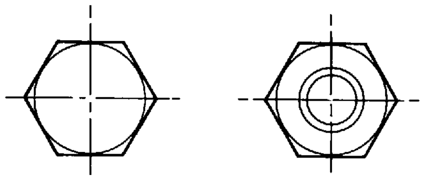
bolt | nut/washer

1 4.6 — black mild steel



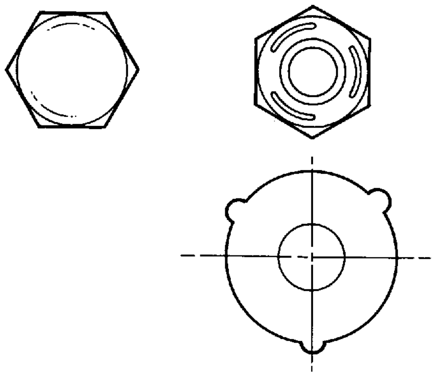
BS4190 BS4190

2 8.8 — high-tensile steel



BS 3693 BS 3692

3 high-strength friction-grip bolts general grade, part 1



4 high-strength friction-grip bolts higher grade, part 2