Manual for the design of steelwork building structures

This *Manual* provides guidance on the design of single and multi-storey building structures using structural steelwork. Structures designed in accordance with this *Manual* will normally comply with BS 5950-1: 2000 as amended to March 2008.

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

- braced multi-storey structures that do not rely on bending resistance of columns for their overall stability and are classified as non-sway
- single-storey structures using portal frames or posts and lattice trusses.

The *Manual* provides:

- 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.

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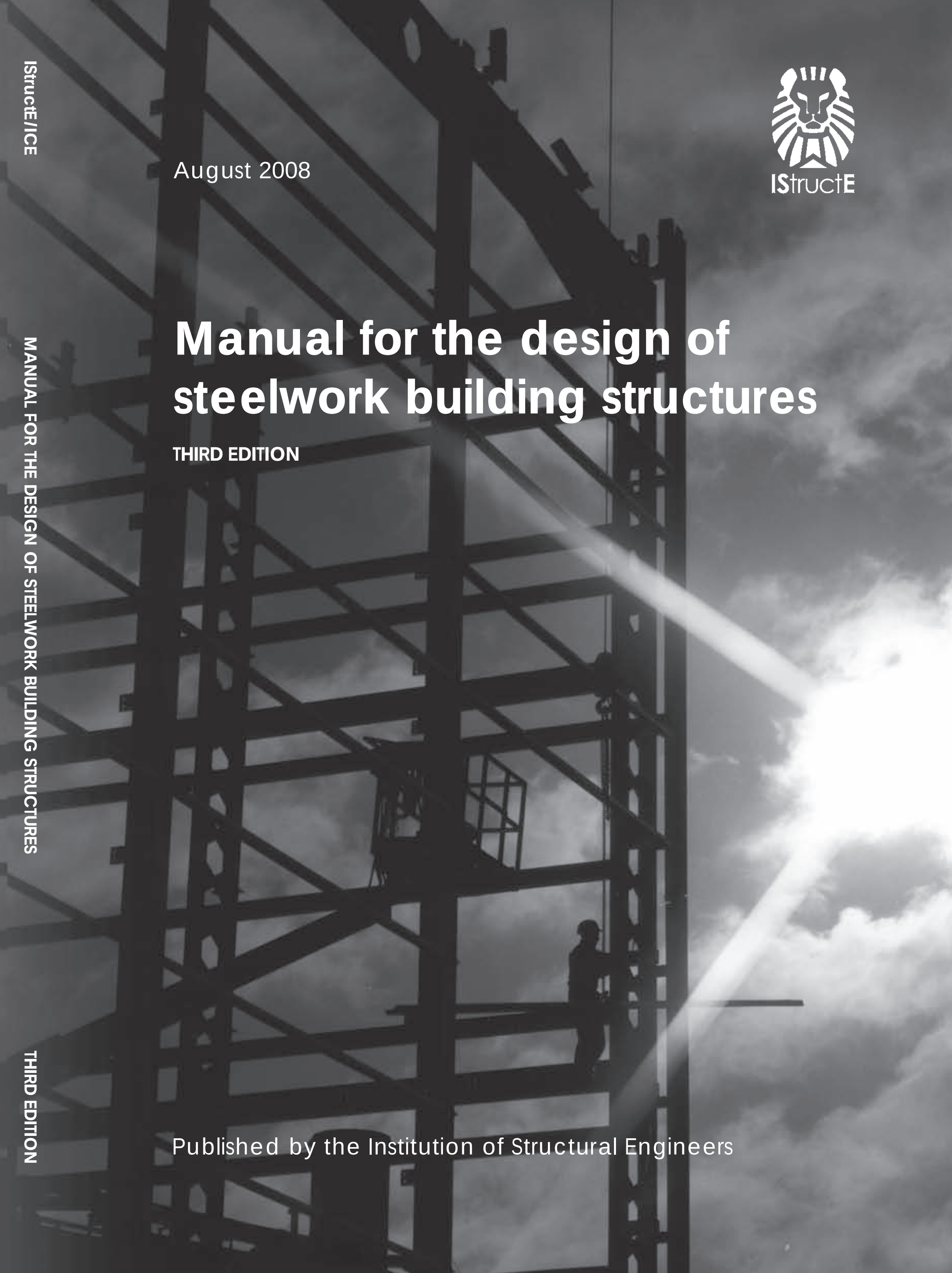
MANUAL FOR THE DESIGN OF STEELWORK BUILDING STRUCTURES
THIRD EDITION



August 2008

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THIRD EDITION

Published by the Institution of Structural Engineers

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The Institution of Civil Engineers

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Contents

1 Introduction

1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of single and multi-storey building structures using structural steelwork. Structures designed in accordance with this *Manual* will normally comply with BS 5950-1:2000[1] (as amended to March 2008).

1.2 Scope of the *Manual*

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

- braced multi-storey structures that:
  - do not rely on bending resistance of columns for their overall stability, and
  - are classified by BS 5950-1:2000[1] as 'non-sway' (see below)
- single-storey structures using portal frames, posts and lattice trusses or posts and pitched roof trusses.

A structure may be classified as 'non-sway' if its sway deformation is sufficiently small for the resulting secondary forces and moments to be negligible. For clad structures, provided that the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting is not explicitly taken into account, this may be assumed to be satisfied if the sway mode elastic critical load factor λ_{cr} of the frame, under vertical load only, satisfies:

$$\lambda_{cr} \geqslant 10$$

BS 5950-1:2000 provides guidance on the determination of λ_{cr}

For some situations, BS 5950-1:2000 provides more than one method of calculation. These include more elaborate methods not given in the *Manual*. Such methods are generally more rigorous and more accurate than the methods adopted in the *Manual*.

For structures outside this scope, BS 5950-1:2000 and the remaining parts of BS 5950 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 *Steelwork design: guide to BS 5950-1:2000, Volume 1*[2] (the 'blue book') published by SCI.

2 General principles

2.1 General

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

One Structural 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 should not sway, deflect or vibrate more than the specified limits. Consideration should be given to the erection procedure and stability during construction. In the UK, the Construction (Design and Management) Regulations[3] place significant responsibilities on the designer, with the aim of ensuring that structures can be constructed, used, maintained and eventually demolished in a safe manner.

2.2 Designing for safety

Whether enshrined in law, statutory regulation, contractual requirements, good practice or moral code, designing for safety is recognised as an absolute requirement.

Important principles which should be implemented when carrying out project design work are:

- identification of the designer’s absolute duties, and those where the designer has to take reasonable steps
- identification of design responsibilities that lie within the construction chain, from client to subcontractor to user
- hazard elimination and risk reduction (eliminate, reduce, inform, control)
- the fundamental need for salient information particularly at sensibility or contractual interfaces.

These principles are exemplified in the UK in the Construction (Design and Management) Regulations 2007[3], which is supported by developed central[4] and industry guidance[5], all of which are valuable references that warrant consideration.

2.3 Stability

2.3.1 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
- vertical braced bays in conjunction with plan bracing.

2.3.2 Multi-storey 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.

2.3.3 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
  - cantilever columns from moment resistant base
  - reinforced concrete walls preferably not less than 180mm 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
  - uplift forces generated from the vertical bracing system shall be adequately (safely) held down by the foundations.

2.4 Robustness

All members of a structure should be effectively tied together in the longitudinal, transverse and vertical directions as set out in Sections 4 and 12. 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, see BS 5950-1:2000[1] clause 2.4.5.4 and BS 6399-1:1996[6] clause 12.

2.5 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 approximately equal 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 60m 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.

2.6 Load combinations

The following load combinations should normally be considered in design:

- Combination 1: Dead load plus imposed gravity loads
- Combination 2: Dead load plus wind load
- Combination 3: Dead load plus imposed load plus wind load.

Structures subject to loading from overhead travelling cranes are beyond the scope of the *Manual.*

2.7 Loading

2.7.1 Applied loading

This *Manual* adopts the limit-state principle and the load factor format of BS 5950-1:2000[1]. The unfactored loads to be used in calculations are obtained as follows:

- unfactored dead load, G_k: the weight of the structure complete with finishes, fixtures and fixed partitions (BS 648:1964[7])
- unfactored imposed load, Q_k (BS 6399-1:1996[6], BS 6399-3:1988[8])
- roof loading Q_k (BS 6399-3:1988[8])
- unfactored wind load, W_k (BS 6399-2:1997[9]).

2.7.2 Additional horizontal loading

Additional horizontal loading is considered in design to ensure the structure is robust and capable of resisting the effects of small accidental loads and imperfections in construction. Major accidental effects are covered by the requirements for tying forces and key elements.

Two requirements are present in the code to ensure this robustness.

- The minimum value of the wind force must not be less than 1% of the factored dead load on the structure ($\gamma_f G_k$) applied horizontally.
- A notional horizontal load of 0.5% of the factored dead plus imposed load is applied to the structure at each level, see Fig. 1, in load combination 1.

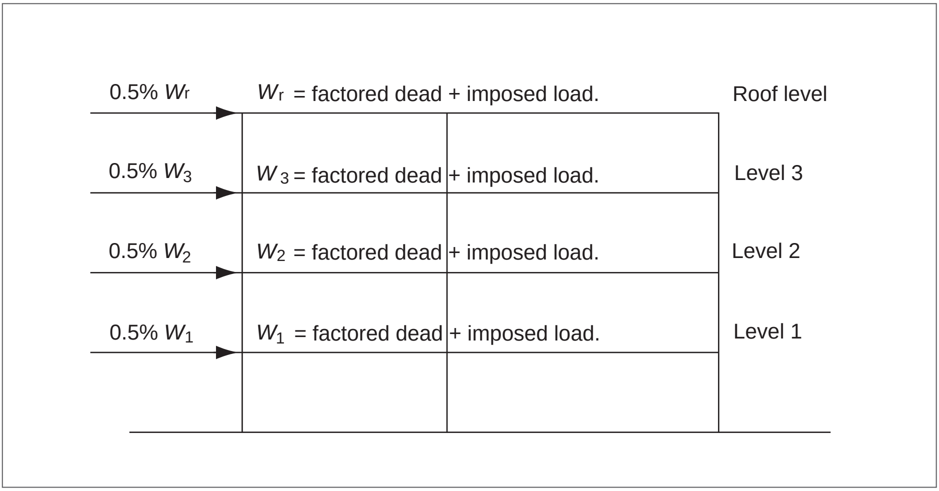


Fig. 1 Load combinations

This notional load is not applied:

- when considering overturning
- when considering pattern loads
- when considering other horizontal loads
- when considering temperature effects
- as a contribution to the net reactions to foundations.

These combinations do not apply to the minimum wind load.

2.8 Limit states

2.8.1 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.

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

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	–	1.0
2 dead + wind	1.4	1.0	–	–	1.4	–
3 dead + wind + imposed	1.2	1.0	1.2	0	1.2	–

2.8.2 Serviceability limit states

2.8.2.1 Deflection

The structure and its members should be checked for deflections under unfactored imposed loads and unfactored wind loads. The deflection should also be checked where necessary for the combined effects of imposed and wind loads using 80% of the unfactored values.

The deflections for beams arising from unfactored imposed loads should normally be limited to the following values, unless smaller limits are required to limit vibration:

- cantilevers — length/180
- beams carrying plaster or other brittle finish — span/360
- all other beams (excluding purlins and sheeting rails) — span/200

The deflections of purlins and side rails should be limited to suit the characteristics of the particular cladding.

In certain circumstances an upper limit on the total deflection due to both dead and imposed loads can apply.

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

- columns in all single-storey buildings, except portal frames — height/300
- columns in multi-storey buildings — height of storey/300

For some buildings other values than those shown above may be more appropriate.

In particular for multi-storey buildings a ratio of height of storey/500 may be more suitable where the cladding cannot accommodate larger movements. In all cases the deflection of columns and beams in a cladding system should be restricted to prevent damage to the cladding.

2.8.2.2 Corrosion protection

Structural steel members often need 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. Further information is available from BS EN ISO 12944[10], *Steelwork corrosion control*[11] and Corus[12].

2.8.2.3 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 20mm. Reference may also be made to the SCI publication, *Design of floors for vibration*[13]. Floors supporting sensitive equipment or subject to dancing etc. may need special consideration.

2.8.3 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 5950-8:2003[14] should be consulted. Reference should also be made to *Fire protection for structural steel in buildings*[15].

Care must be exercised in the selection of fire protection materials to ensure that they are sufficiently robust to resist damage during their service life and in the event of a fire. It is useful to clarify the responsibility for the design of the fire protection aspects within the design team at an early stage.

2.9 Material properties

2.9.1 Design strength p_y

This *Manual* covers the design of structures fabricated from steels supplied to BS EN 10025-1:2004[16]. Design strengths, p_y should be obtained from Table 2.

Table 2 Design strengths, p_y

BS EN 10025:1993[16] and BS EN 10210-1:1994[17]	Thickness less than or equal to (mm)	Sections, plates, hollow sections (N/mm²)
S275	16	275
	40	265
	63	255
	80	245
	100	235
S355	16	355
	40	345
	63	335
	80	325
	100	315

For other steels not covered by Table 2, reference should be made to BS 5950-1:2000[1].

2.9.2 Brittle fracture

Reference should be made to BS 5950-1:2000[1].

2.9.3 Modulus of elasticity

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

2.9.4 Coefficient of linear expansion

The coefficient of linear expansion, α, should be taken as 12 x 10^{-6} per $^\circ$C.

3.1 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 two conditions are identified:

Condition I — Full lateral restraint provided (e.g. beams supporting concrete floors). This condition will be satisfied if the connection between the compression flange of the member and the floor it supports is capable of resisting a lateral force of at least 2.5% of the force in the compression flange arising from the factored loads.

Condition II — Full lateral restraint not provided.

The design procedures are described separately below for both conditions.

3.2 Condition I: Full lateral restraint provided

Design procedure

a) Calculate the factored load 1.6 x imposed + 1.4 x 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 Section 2.8.2 for simply supported beams:

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

where
- I is the second moment of area required (cm^4)
- W is the total unfactored imposed distributed or point load (kN)
- L is the span (m)
- C is the deflection coefficient obtained for each loading from Fig. 2

When more than one load is imposed on the beam the principle of superposition may be used. For cantilevers with backspans, 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 (up or down at the tip).

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 $\geqslant$ than the maximum moment on the section 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 Tables 11 and 12 of BS 5950-1:2000[1] and that different section modulii are used for calculating the moment capacities for different classes of sections.

In the blue book[2] 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 3 have been abstracted from the blue book.

Table 3 Section classification for bending only

All equal flanged rolled sections (UB, UC and joists) and all parallel flange channels are plastic or compact for bending about the major axis except as given below. For CHS, SHS and RHS see the blue book[2].

Grade S275 steel, semi-compact sections	Grade S355 steel, semi-compact sections
356 x 368 x 129 UC	356 x 171 x 45 UB
152 x 152 x 23 UC	356 x 368 x 153 UC
	356 x 368 x 129 UC
	305 x 305 x 97 UC
	254 x 254 x 73 UC
	203 x 203 x 46 UC
	152 x 152 x 23 UC

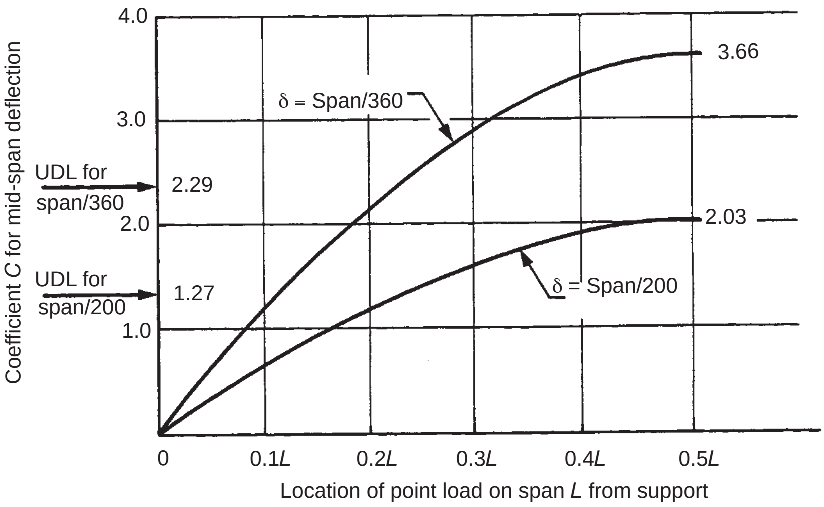


Fig. 2 Deflection coefficient C for simply supported beams

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

M_{cx} $= p_y S_x$, but $\leqslant 1.2 p_y Z_x$ for plastic or compact sections
$= p_y Z_x$ for semi-compact

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
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 previously.

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

For slender sections an effective section is necessary, and BS 5950-1:2000[1] 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 $P_v = 0.6 p_y A_v$ where p_y is obtained from Table 2, and A_v is the shear area defined as follows:

- rolled I, H and channel sections, load parallel to web = tD
- rectangular hollow sections, load parallel to webs = $AD/(D+B)$
- rolled tee sections, load parallel to web = tD
- circular hollow sections = $0.6A$
- solid bars and plates = $0.9A$
- any other case refer to BS 5950-1:2000[1]

where t is the web thickness
D is the overall depth of the section
B is the overall breadth of the section
A is the area of the cross-section

Alternatively P_v may be obtained from the blue book.

For rolled sections, if the depth of the web d is greater than $70t\varepsilon$ the web should be checked for shear buckling – refer to BS 5950-1:2000[1]. However, none of the common rolled sections listed in Appendix A exceed this limit.

No further checks are required if the shear force $F_v \leqslant 0.6P_v$

Where $F_v > 0.6P_v$ 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. For symmetrical sections the reduced moment capacity in the presence of high shear M_{cv} may be obtained from the simplified formula for plastic and compact sections:

$$M_{cv} \leqslant M_{cx} - \left[2\left(\frac{F_v}{P_v}\right) - 1.0\right]^2 \frac{p_y D^2 t}{4}$$

In all other cases, reference should be made to BS 5950-1:2000[1].

Note that this reduction starts at $F_v/P_v = 0.5$, but the reduction is very small unless F_v/P_v is greater than 0.6.

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 5950-1:2000[1] or the tables in the blue book should be used.

3.3 Condition II: Full lateral restraint not provided

All beams designed by this method should also satisfy the requirements of Condition I for bending, deflection, shear, web bearing and buckling. The guidance below is applicable only to simply supported beams and to cantilevers without intermediate restraints. For continuous beams and other members subject to double curvature bending, and for cantilevers with intermediate restraints, reference should be made to BS 5950-1:2000[1].

Design procedure

a) Calculate the factored load = 1.6 x imposed + 1.4 x 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 Section 2.8.2. For simply supported beams, use the method described in Section 3.2b).

c) Determine the effective length L_E from the following two cases.
- Beams with lateral restraints at their ends only. The effective length L_E should be obtained from Table 4 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 5. If a bending moment is applied at the tip of the cantilever, the effective length should then be increased by the greater of 30% or $0.3L$.
- 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, L_{LT} multiplied by an appropriate factor (1.0 L_{LT} for normal loading conditions or 1.2 L_{LT} for destabilizing loading conditions).

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 4 for the conditions of restraint at the support, taking L_{LT} 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. Intermediate restraints provided at intervals along a beam should be capable of resisting a total force of not less than 2.5% of the maximum factored force in the compression flange divided between the restraints in proportion to their spacing. However, where three or more intermediate restraints are provided, each individual restraint should be capable of resisting not less than 1% of the maximum factored force in the compression flange.

When a series of two or more parallel beams require 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 an independent robust part of the structure or held in a fixed relationship to each other by means of triangulated plan bracing.

The bracing system should be capable of resisting each of:

- the 1% restraint force considered as acting at one point at a time
- the 2.5% restraint force divided between the intermediate restraints in proportion to their spacing summed over each of the beams to which the bracing provides restraint but reduced by a factor k_r. The factor is given as:

$$k_r = \left(0.2 + \frac{1}{N_r}\right)^{0.5}$$

where N_r is the number of parallel members restrained.

Table 4 Effective length of beams L_E

Conditions of restraint at the ends of the beams		Loading conditions	
		Normal	**Destabilizing**[a]
Compression flange laterally restrained: nominal torsional restraint against rotation about longitudinal axis[b]	Both flanges fully restrained against rotation on plan	$0.7L_{LT}$	$0.85L_{LT}$
	Compression flange fully restrained against rotation on plan	$0.75L_{LT}$	$0.9L_{LT}$
	Both flanges partially restrained against rotation on plan	$0.8L_{LT}$	$0.95L_{LT}$
	Compression flange partially restrained against rotation on plan	$0.85L_{LT}$	$1.0L_{LT}$
	Both flanges free to rotate on plan	$1.0L_{LT}$	$1.2L_{LT}$
Compression flange laterally unrestrained; both flanges free to rotate on plan	Restraint against torsion about the longitudinal axis provided by positive connection of bottom flange to supports	$1.0L_{LT} + 2D$	$1.2L_{LT} + 2D$
	Partial torsional restraint against rotation about the longitudinal axis provided only by pressure of bottom flange on supports	$1.2L_{LT} + 2D$	$1.4L_{LT} + 2D$

Nomenclature

D is the depth of the beam

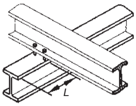
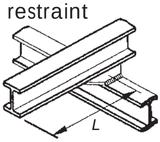
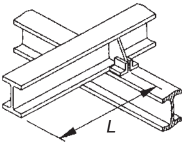
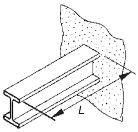
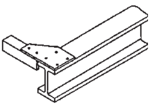
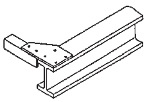
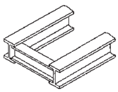
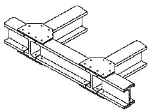
L_{LT} is the length of the beam between its ends

Notes

a 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.

b Nominal torsional restraint may be supplied by web cleats, partial depth end plates or fin plates.

Table 5 Effective length of cantilevers L_E

Restraint conditions		Loading conditions	
At support	**At tip**	**Normal**	**Destabilizing**
a) Continuous, with lateral restraint to top flange 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) Continuous, with partial torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) Continuous, with lateral and torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) Restrained laterally, torsionally and against rotation on plan 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L
Tip restraint conditions			
1) Free  (not braced on plan)	2) Lateral restraint to top flange  (braced on plan in at least one bay)	3) Torsional restraint  (not braced on plan)	4) Lateral and torsional restraint  (braced on plan in at least one bay)

d) Choose a trial section and grade of steel and check that the factored moment M_x on any portion of the beam between adjacent restraints, does not exceed the buckling resistance moment M_b of the section chosen divided by the equivalent uniform moment factor m_{LT}

where m_{LT} is the equivalent uniform moment factor obtained from Table 6
M_x is the maximum on the portion of the member being considered

The buckling resistance moment M_b is obtained from $M_b = p_b S_x$, only for plastic or compact sections. For semi-compact or slender sections refer to BS 5950-1:2000[1].

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

The equivalent slenderness λ_{LT} is given by:

$$\lambda_{LT} = uv\lambda$$

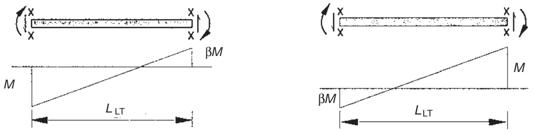
where λ is the effective length L_E obtained as described in Section 3.3c) divided by the radius of gyration r_y of the chosen section about its minor axis
u is the buckling parameter which may be taken as 0.9 for equal flanged I-, H- and channel sections, or may be obtained from section properties tables in the blue book[2]
v is a slenderness correction factor which may be obtained from Table 7 for all symmetric equal flanged members for the appropriate value of λ/x or Table 19 of BS 5950-1:2000[1] for all other sections. λ/x is obtained from λ determined as above and x is the torsional index which may be taken as D/T (provided u is taken as 0.9) or more accurately from section tables

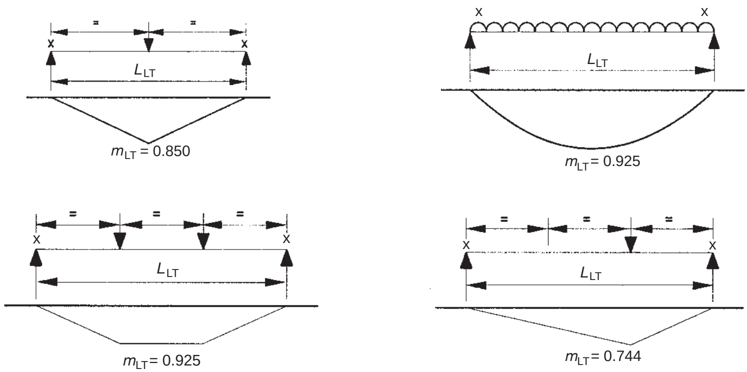
e) Check that the beam complies with the requirements for bending and deflection using the procedure detailed in Section 3.2b) and c) with due allowance for the effects of high shear on the moment capacity as indicated in Section 3.2d).

f) Check that the shear capacity P_v of the sections exceeds the factored shear forces (F_v) using the procedure detailed in Section 3.2d).

g) Check for web bearing and buckling as detailed in Section 3.2e).

Table 6 Equivalent uniform moment factor, m_{LT}

Bending moment diagram between restraints m_{LT}	β	m_{LT}
β positive 	1.0	1.00
	0.9	0.96
	0.8	0.92
	0.7	0.88
	0.6	0.84
	0.5	0.80
	0.4	0.76
	0.3	0.72
x = lateral restraint	0.2	0.68
	0.1	0.64
β negative 	0.0	0.60
	-0.1	0.56
	-0.2	0.52
	-0.3	0.48
	-0.4	0.46
	-0.5 to -1.0	0.44



Nomenclature

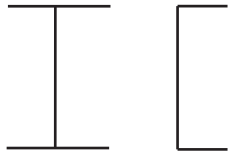
β is the ratio of the smaller end moment to the larger

Notes

a For cantilevers without intermediate restraint, members subject to destabilizing loads, continuous members with unequal flanges and tapered sections, m_{LT} = 1.0.

b For the general case refer to BS 5950-1:2000[1].

Table 7 Slenderness factor, *v* for flanged beams of uniform section[a]



λ/x	v
0.5	1.00
1.0	0.99
1.5	0.97
2.0	0.96
2.5	0.93
3.0	0.91
3.5	0.89
4.0	0.86
4.5	0.84
5.0	0.82
5.5	0.79
6.0	0.77
6.5	0.75
7.0	0.73
7.5	0.72
8.0	0.70
8.5	0.68
9.0	0.67
9.5	0.65
10.0	0.64
11.0	0.61
12.0	0.59
13.0	0.57
14.0	0.55
15.0	0.53
16.0	0.52
17.0	0.50
18.0	0.49
19.0	0.48
20.0	0.47

Note

a For other sections refer to BS 5950-1:2000[1]

3.4 Cased beams

3.4.1 Introduction

This Section describes the design of cased beams that are subject to bending only and which satisfy the conditions in Section 3.4.2. The design of cased beams not satisfying these conditions should be carried out by reference to BS 5950-1:2000[1]. To allow for the additional stiffening afforded by the concrete casing the design should be carried out by following the design procedure described in Section 3.4.3.

3.4.2 Conditions

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

- the steel section is either:
  - a single rolled section or a fabricated section with equal I- or H-flanges, or
  - rolled equal channel sections arranged in contact back to back or separated by a minimum gap of 20mm and a maximum gap of half the depth of the section. In order to act together the channels must be interconnected, refer to BS 5950-1:2000
- the dimensions of the steel sections do not exceed a depth of 1000mm (parallel to the web(s)) or a width of 500mm
- the steel section is unpainted and is free from oil, grease, dirt and loose rust and millscale
- there is a minimum rectangle of concrete casing consisting of well compacted ordinary dense concrete of at least grade 25 to BS 8110-1:1997[18] and extending the full length of the steel member and its connections
- the concrete casing may be chamfered at corners but should provide cover to the outer faces and edges of at least 50mm
- the casing is reinforced with either:
  - D98 fabric complying with BS 4483:2005[19], or
  - a cage of closed links and longitudinal bars using steel reinforcement or wire not less than 5mm diameter and complying with BS 4449:2005[20] or BS 4482:2005[21], 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 8110-1:1997.
- the effective length L_E of the cased section is limited to the least of
  $40b_c$ or $(100b_c^2/d_c)$ or $250r$

  where b_c and d_c are, respectively, the minimum width of solid casing within the depth of the steel section and the minimum depth of solid casing within the width of the steel section

  r is the minimum radius of gyration of the uncased steel section

3.4.3 Design procedure

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

- the second moments of area I_{cs} for the cased section for calculations of deflection may be taken as:

$$I_s + \frac{(I_c - I_s)E_c}{E}$$

where I_s is the second moment of area of the steel section
I_c is the second moment of area of the gross concrete section
E is the modulus of elasticity of steel
E_c is the modulus of elasticity for the relevant grade of concrete, see BS 8110-:1997[18].

- in the calculations of slenderness, the radius of gyration of the cased section should be taken as the greater of:
  - $0.2b_c$ but $\leqslant 0.2\,(B + 150)$ mm, or
  - r_y of the uncased section

  where b_c is the minimum width of the solid concrete casing within the depth of the steel section
  B is the width of the steel flanges
- the buckling resistance moment M_b of the cased section should not exceed the moment capacity M_c of the uncased section nor 1.5 times the value of M_b for the uncased section.

3.5 Single equal angles

Design procedure

Equal angles with $b/t \leqslant 15\varepsilon$ 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 follows:

where the heel of the angle is in compression:

$M_b = 0.8p_yZ_x$

where the heel of the angle is in tension:

$$M_b \leqslant \left[\frac{1350\epsilon - \dfrac{L_E}{r_v}}{1625\epsilon}\right] \text{but } M_b \leqslant 0.8p_yZ_x$$

where Z_x is the smaller elastic modulus about the x-x axis
r_v is the radius of gyration about the v-v axis
L_E is the is the effective length based on the length L_v between restraints against buckling about the v-v axis
p_y is the design strength.

For unequal angles see BS 5950-1:2000[1].

3.6 Hollow sections

Design procedure

The procedure given in Section 3.2 for Condition I (full lateral restraint provided) may be followed for square sections (*D/B* = 1) and circular hollow sections and other rectangular sections provided that λ (i.e. L_E/r_y) is within the limits in Table 8, where *D* and *B* are overall depth and breadth of the box section, respectively.

Table 8 Limiting values of λ

D/B	λ	*D/B*	λ	*D/B*	λ
1.25	770 x (275/p_y)	1.50	515 x (275/p_y)	2.00	340 x (275/p_y)
1.33	670 x (275/p_y)	1.67	435 x (275/p_y)	2.50	275 x (275/p_y)
1.40	580 x (275/p_y)	1.75	410 x (275/p_y)	3.00	225 x (275/p_y)
1.44	550 x (275/p_y)	1.80	395 x (275/p_y)	4.00	170 x (275/p_y)

4.1 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. This aim has also to embrace the principle of designing for safety expressed in Section 2.2.

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.

4.2 Loads

Loads should be based on BS 648:1964[7] and BS 6399 -1:1996[6], -2:1997[9] and -3:1988[8].

4.2.1 Imposed loading, Q_k

Imposed loading is specified in BS 6399-3:1988[8] as a minimum of 0.60kN/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 1.5kN/m^2. BS 6399-3:1988 also gives the loads arising from the effects of uniformly distributed and drifted snow.

4.2.2 Wind loading, W_k

Wind loading is specified in BS 6399-2:1997[9] and varies with the roof pitch, location of the building, topography, etc. In order to maintain stability BS 5950-1:2000[1] requires that the horizontal wind load should be not less than 1% of the factored dead load. Consideration should be given to the number and position of door openings in external walls and their possible effect on overall wind loading effects on the structure.

4.2.3 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.

In the absence of firm details the initial design may use the following typical loads:

- roof sheeting and side cladding 0.1 to 0.2kN/m^2
- steelwork 0.1 to 0.2kN/m^2
- ceiling and services 0.1 to 0.3kN/m^2

The initial load assumptions must be verified as part of the detailed design.

4.2.4 Notional loading, N_k

Notional horizontal load, N_k at each level should be:

$$0.5\% \ (1.4G_k + 1.6Q_k)$$

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

4.2.5 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. 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.

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

In addition to the load combinations given in Table 1, single storey buildings should also be designed for avoidance of disproportionate collapse. For guidance on designing for robustness see Section 12.

4.3 Material selection

In the UK, grade S275 steel should be used generally for rafters and columns, although grade S355 may be used unless deflection is likely to be critical. Grade S355 steel may be used for latticed or trussed members. Grade 8.8 bolts should normally be used throughout, preferably all the same size.

4.4 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:

- the following spacings between frames and spans, which are likely to be economic:

		spacing (m)	span (m)
–	portals	5.0 – 7.0	up to 60[22]
–	post-and-pitched truss	4.5 – 7.5	18 – 25
–	post-and-lattice girder	4.5 – 7.5	20 – 40

- a choice of structural framing between functional, architectural, economic and consideration of client requirements
- for portal frames provide longitudinal stability against horizontal forces by placing vertical bracing in the side walls deployed symmetrically wherever possible
- 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
- provide bracing in the roof plane of all single-storey construction to transfer horizontal loads to the vertical bracing
- 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
- consider the provision of movement joints for buildings in the UK whose plan dimensions exceed 70m
- 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
- 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
- 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
- provide framing to openings to transfer horizontal forces to braced elements
- at the ends of the building gable frames and openings must be suitably arranged and braced to ensure they are stable and can transfer door and wind loadings to appropriate supports.

4.5 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 columns. In the latter case, the stability of the column should be justified as a cantilever in fire conditions, see references 23, 24 and 25. For the column to be considered as a cantilever it needs to have a fixed base. Fire protection should also be considered for structural members supporting mezzanine floors enclosed by single-storey buildings.

4.6 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:

- steelwork integral with external cladding and that which is not readily accessible for inspection and maintenance: by concrete encasement, or high-quality corrosion protection (i.e. galvanizing and bituminous paint, etc)
- internal steelwork: by a protective system commensurate with the internal environment
- external steelwork: by a protective system commensurate with the external environment.

For different environments and for more detailed advice reference should be made to appropriate British Standards and to publications from Corus, the British Constructional Steelwork Association, the Zinc Development Association and the 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.

4.7 Bracing

Choose the location and form of bracing in accordance with the recommendations in Sections 2.3.3 and 4.4. Typical locations are shown on Figs. 3 and 4 for single-storey buildings.

The wind load or the notional horizontal forces on the structure, whichever are greater, should be assessed for the appropriate load combinations and distributed to the bracing bays resisting the horizontal forces in each direction in a realistic manner. For symmetrically placed bracing bays of approximately equal stiffness, the distribution can be in equal proportions.

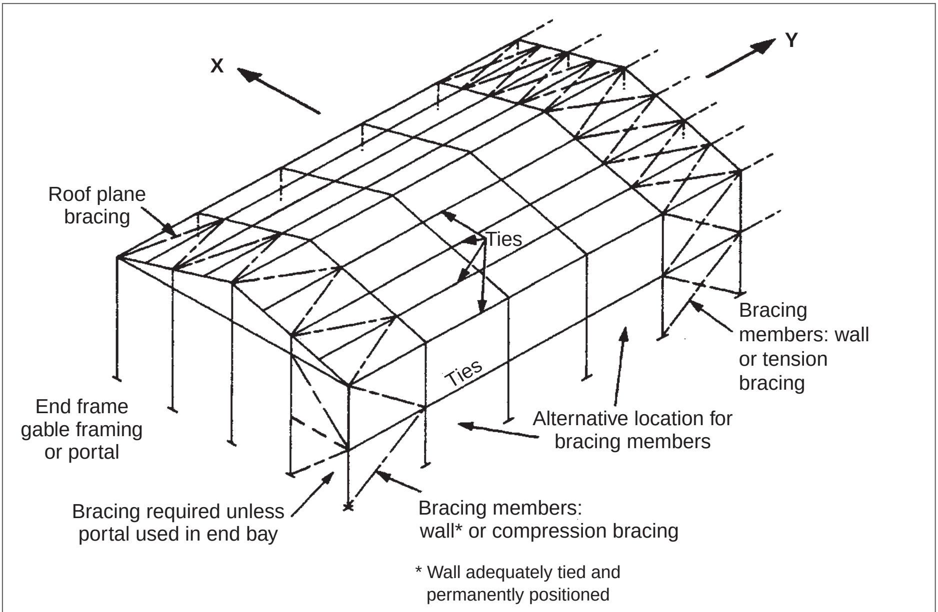
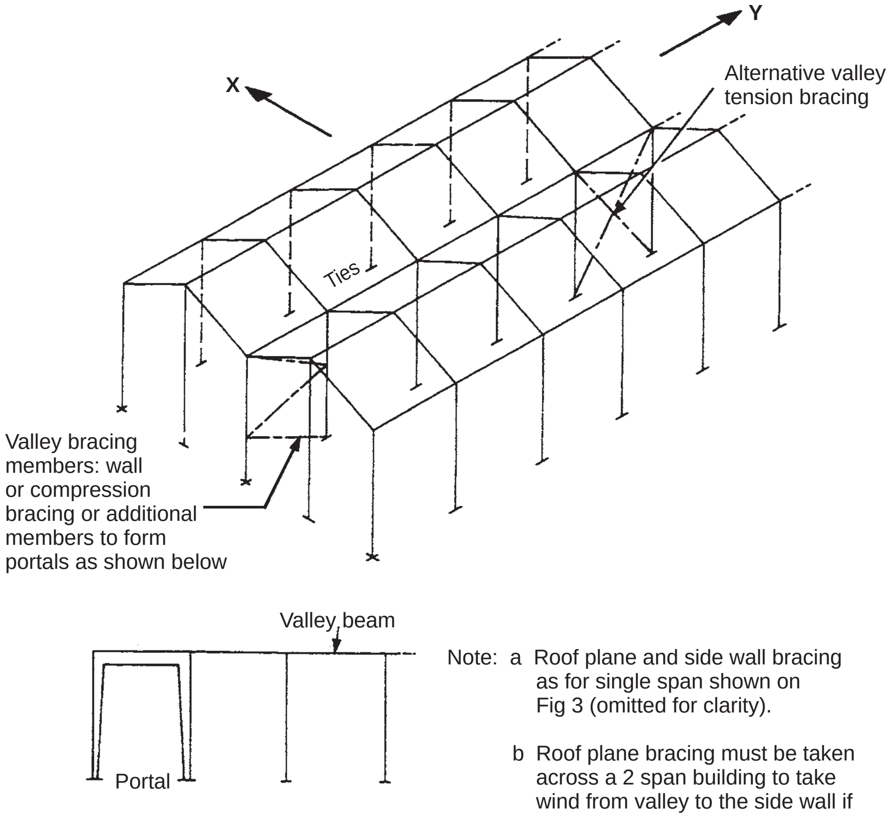


Fig. 3 Single-span portals



Note: a Roof plane and side wall bracing as for single span shown on Fig 3 (omitted for clarity).

b Roof plane bracing must be taken across a 2 span building to take wind from valley to the side wall if valley bracing is to be omitted. Such an arrangement should be limited to 2 span buildings.

Fig. 4 Multi-span portals

4.8 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.

Table 9 Lightweight roofing systems and their relative merits					
Description	**Minimum pitch**	**Typical depth (mm)**	**Typical span (mm)**	**Degree of lateral restraint to supports**	**Comments**
Galvanised corrugated steel sheets	10°	75 Sinusoidal profile	1800 –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 – 88	925 – 1800	Fair	Industrial and agricultural buildings. Brittle construction. Usually fixed to purlins with hook bolts
Profiled aluminium sheeting (insulated or uninsulated)	6°	30 – 65	1200 –3500	Good if fixed direct to purlins	Good corrosion resistance but check fire requirements and bi-metallic corrosion with mild steel supporting members
Profiled coated steel sheeting (insulated or uninsulated)	6°	25 – 65	1500 –4500	Good if fixed direct to purlins	The most popular form of lightweight roof cladding used for industrial type buildings. Wide range of manufacturers, profile types and finishes
'Standing seam' roof sheeting (steel or aluminium)	2°	45	1100 –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
Galvanised Steel or Aluminium Decking Systems	Nominally flat	32 – 100	1700 –6000	Very good	Used for flat roof with insulation and vapour barrier and waterproof membrane over. Fire and bi-metallic corrosion to be checked if aluminium deck used
Timber	Nominally flat	General guidance as for timber floors			
Reinforced woodwool slabs	Nominally flat	50 – 150	2200 – 5800	Good if positively fixed to beam flanges	Pre-screeded type can prevent woodwool becoming saturated during construction

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 9 which summarizes the salient features of the various types of lightweight roofing systems commonly used in the UK for single-storey structures.

In general, most of the profiled decking systems described in Table 9 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 columns to support such brick panel walls against lateral loading.

5.1 Purlins

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

5.1.1 Cold-formed sections

For cold-formed sections empirical rules and design formulae are given in BS 5950-5:1998[26]. 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.

5.1.2 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 fixings thereof to be capable of providing lateral restraint to the purlins
- grade of steel to be a minimum of S275
- unfactored loads to be considered and loading to be substantially uniformly distributed. Not more than 10% of the total roof load on the purlin should be due to other types of load
- span not to exceed 6.5m, and roof pitch not to exceed 30°
- single span purlins to be connected at each end by at least two bolts. If the purlins are generally continuous over two or more bays, with staggered joints in adjacent lines of purlins, single bay members should have at least one end connected by two or more bolts.

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, should be not less than the larger of the two values, Z_p and Z_q given in Table 10. In addition, the dimensions D and B, depth and width respectively, should be not less than the values given in Table 10.

Table 10 Limiting values for purlin design

Section	Z_q (cm³) (Wind load from BS 6399-2:1997[9])	Z_p (cm³)	D (mm)	B (mm)
Angles	$\frac{W_p L}{1800}$	$\frac{W_q L}{2250}$	$\frac{L}{45}$	$\frac{L}{60}$
CHS	$\frac{W_p L}{2000}$	$\frac{W_q L}{2500}$	$\frac{L}{65}$	-
RHS	$\frac{W_p L}{1800}$	$\frac{W_q L}{2250}$	$\frac{L}{70}$	$\frac{L}{150}$

Nomenclature

W_p and W_q are the total unfactored loads, in kN, on one span of the purlin due to (dead plus imposed) and (wind minus dead) respectively acting perpendicularly to the plane of the cladding

L is the span of the purlin, in mm. However if properly supported sag rods are used L may be taken as the sag rod spacing in determining B only

5.2 Side rails

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

5.2.1 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.

5.2.2 Angles and hollow sections

Angle and hollow-section side rails 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 restraint to the side rails
- grade of steel to be a minimum of S275
- unfactored loads to be considered and the loading to consist generally of wind and selfweight of cladding. Not more than 10% of the total load on the side rail should be due to other types of load or to loads that are not uniformly distributed
- span not to exceed 6.5m, and the slope of the cladding not to exceed 15°
- single span side rails to be connected at each end by at least two bolts. If the side rails are generally continuous over two or more bays, with staggered joints in adjacent lines of side rails, single bay members should have at least one end connected by two or more bolts.

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 those given in Table 11.

Table 11 Limiting values for side rail design

Section	Z_1 (cm³) (Wind load from BS 6399-2:1997[9])	Z_2 (cm³)	D (mm)	B (mm)
Angles	$\frac{W_1 L}{2250}$	$\frac{W_2 L}{1200}$	$\frac{L}{45}$	$\frac{L}{60}$
CHS	$\frac{W_1 L}{2500}$	$\frac{W_2 L}{1350}$	$\frac{L}{65}$	-
RHS	$\frac{W_1 L}{2250}$	$\frac{W_2 L}{1200}$	$\frac{L}{70}$	$\frac{L}{100}$

Nomenclature

W_1 and W_2 are the total unfactored loads, in kN, on one span of the side rail, acting perpendicular to the plane of the cladding and acting parallel to the plane of the cladding respectively

L is the span of the side rail, in mm, taken as follows:
for Z_1 and D the span is centre to centre of the vertical supports
for Z_2 and B the span centre to centre of vertical supports, except that where properly supported sag rods are used L may be as the sag rod spacing

6.1 Introduction

Plastic methods are commonly used for the design of portal frames, resulting in relatively slender structures. In-plane and out-of-plane stability of both the frame as a whole and the individual members must be considered.

The *Manual* gives equations so that in-plane frame stability can be checked by hand but these must only be used in those cases where gravity loading is critical. Wind loads to BS 6399-2: 1997[9] regularly govern the design of portal frames. It is almost always preferable to use specialist software, although the *Manual's* methods may be useful for preliminary design.

The out-of-plane stability of the frame should be ensured by making the frame effectively non-sway out-of-plane. This will usually require bracing or use of very stiff portal frame action, see Section 4.7.

For elastic design, reference should be made to BS 5950-1:2000[1] and to reference 27.

6.2 Plastic design

Guidance is given on the design of single-span portal frames with nominally-pinned bases and where wind loading does not control the design.

For multi-bay frames of equal spans where the same rafter section is used throughout, the design is almost invariably governed by that for the external bays. The internal columns are subjected to very little bending unless the loading is asymmetrical, and may initially be sized the same as the columns for the single-span case. However, internal columns will not usually be provided with the same restraints as external columns. Member stability should be checked considering the effective restraints actually provided.

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 columns
- 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.

6.3 Single-storey portals – sizing of rafters and columns

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:

- plastic hinges are formed at the bottom of the haunch in the column and near the apex in the rafter, the exact position being determined by the frame geometry

- the depth of the haunch below the rafter is approximately the same as the depth of the rafter
- 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
- 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
- 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
- wind loading does not control design.

Design procedure

The procedure to be adopted is set out as follows, and the various dimensions are shown in Fig. 5.

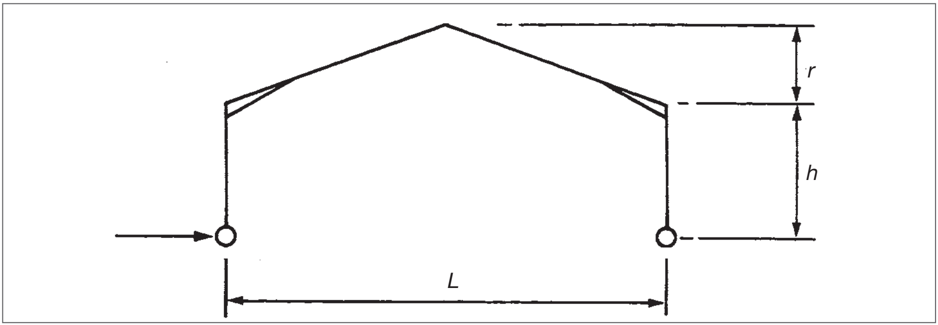


Fig. 5 Dimensions of portal

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 Section 4.2, 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 m^2 and s is the bay spacing)

d) From Fig. 6 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 $H = H_{FR}WL$

f) From Fig. 7 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. 8 obtain the column M_p ratio M_{pl} from r/L and r/h

i) Calculate the Mp required in the column from M_p (column) $= M_{pl}WL^2$

j) Determine the plastic moduli for the rafter S_{XR} and column S_{XL} from

$$S_{XR} = M_p\ (\text{rafter})/p_y$$
$$S_{XL} = M_p\ (\text{column})/p_y$$

where p_y is the design strength obtained from Table 2

Using these plastic moduli, the rafter and column sections may be chosen from the range of plastic sections as so defined in the blue book[2].

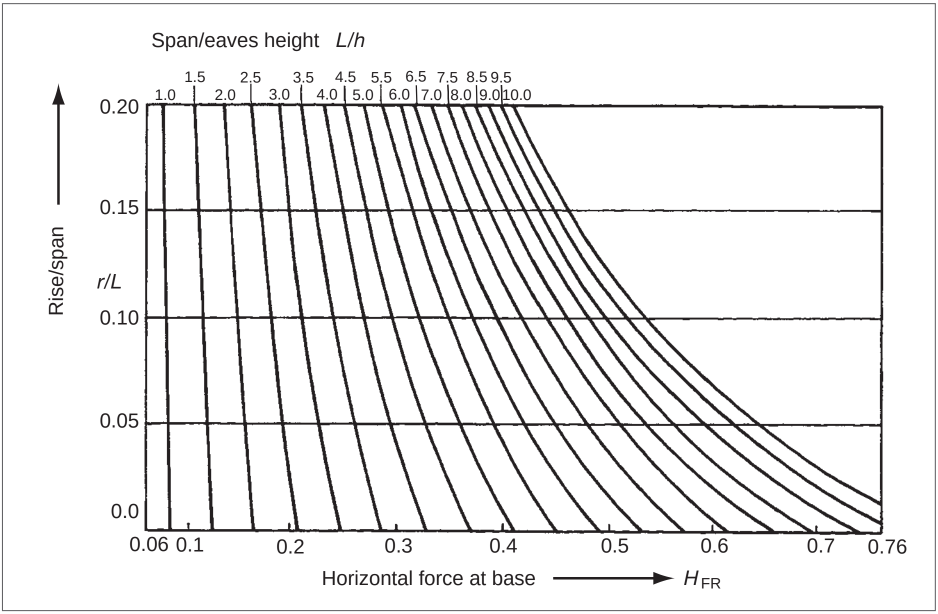


Fig. 6 Horizontal force at base

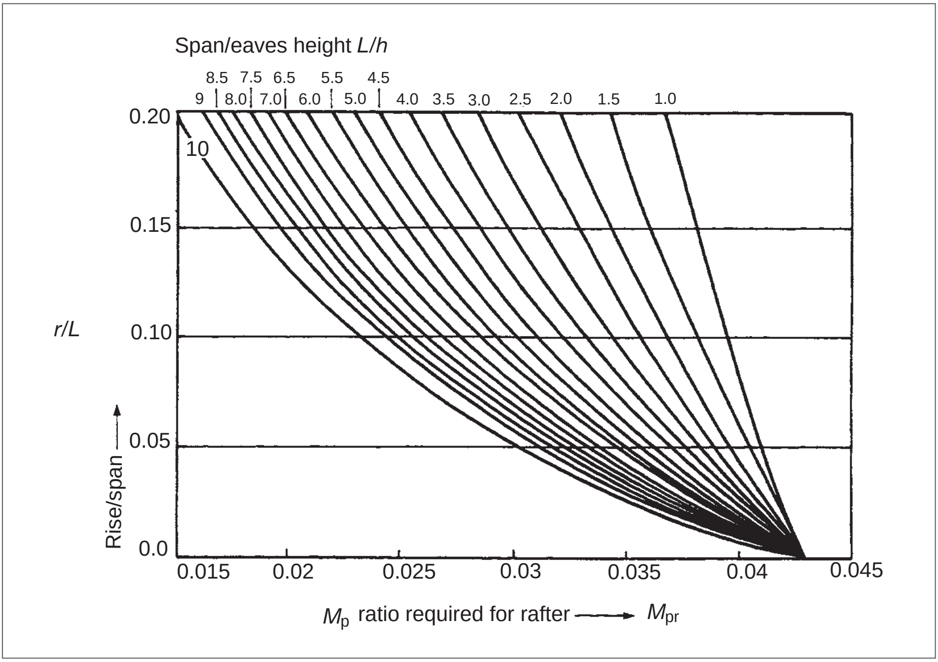


Fig. 7 M_p ratio required for rafter M_{pr}

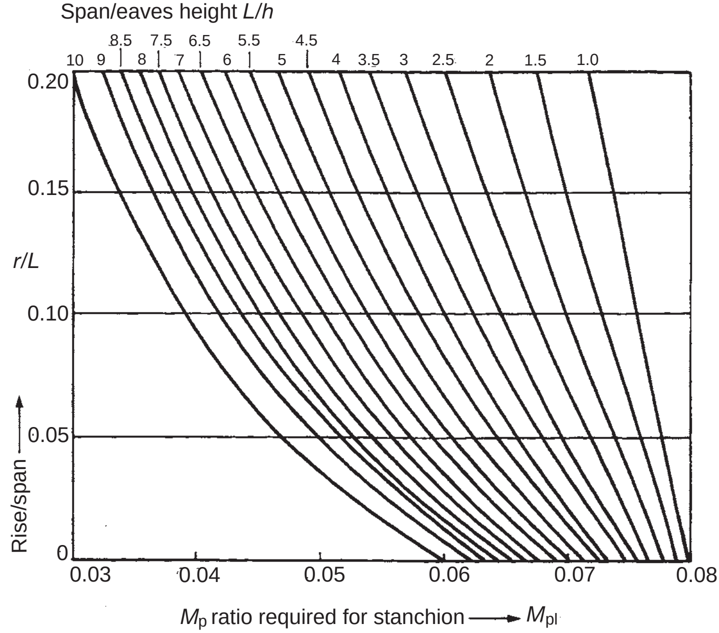


Fig. 8 M_p ratio required for column M_{pl}

6.4 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 rafters in frames of three 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 but must only be used in those cases where vertical loading is critical. Clause 5.5.4 of BS 5950-1:2000[1] gives more elaborate methods of calculation for in-plane stability. If either of the checks given in the *Manual* is not satisfied or vertical loading is not the critical case or the frame does not satisfy one of the other restrictions given in Section 6.4.1, then one of the more elaborate methods should be used. Their use may also avoid the need to adjust member sizes in the case where the equations given in the *Manual* are not satisfied. Further guidance is given in reference 27.

6.4.1 Sway stability check

The method given below may be used only if the frame is not subject to loads from valley beams or crane gantries or other concentrated loads larger than those from purlins. In addition, each bay should satisfy the following conditions:

- the rafter is symmetric about the apex (otherwise refer to BS 5950-1:2000[1])
- the span L does not exceed 5 times the column height h
- the height h_r of the apex above the tops of the columns does not exceed 0.25 times the span L.

If these conditions are satisfied, the in-plane stability may be verified by checking that the ratio L_b/D satisfies the following limit:

$$\frac{L_b}{D} \leqslant \frac{44}{\Omega}\frac{L}{h}\frac{\rho}{\left(4+\frac{\rho L_r}{L}\right)}\frac{275}{p_{yr}}$$

where L_b $= L - [2D_h/(D_s + D_h)]L_h$

ρ $= (2I_c/\ I_r)(L/_h)$ for single-bay frames or

$= (I_c/\ I_r)(L/_h)$ for multibay frames

L is the span of the bay

L_h is the haunch length, see Fig. 9; if the haunches at each side of the bay are different the mean value should be taken

D is the minimum depth of rafters

D_h is the additional depth of the haunch, see Fig. 9

D_s is the depth of rafter allowing for its slope, see Fig. 9

h is the column height

I_c is the second moment of area of column, = 0 if it is not rigidly connected to rafter or if the rafter is supported on a valley beam

I_r is the second moment of area of rafter at its shallowest point

p_{yr} is the design strength of the rafter

Lr is the developed length of rafter of span L, see Fig. 10

Ω $= W_r/W_o$ the ratio of the arching effect of the frame, where

W_r = factored vertical load on rafters

W_o = maximum uniformly distributed vertical load for plastic failure of 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

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 can be adjusted so that the L_b/D condition is satisfied. Alternatively, one of the more elaborate methods given in clause 5.5.4 of BS 5950-1:2000[1] may be used to check whether the original section sizes are adequate.

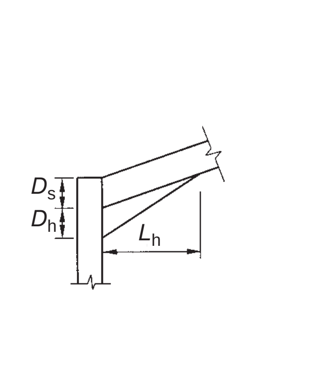


Fig. 9 Dimensions of a haunch

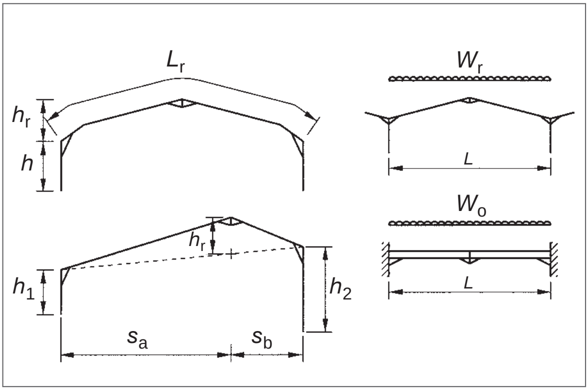


Fig. 10 Portal frame definitions

6.4.2 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 columns and inversion of the rafter. To prevent this the rafter slenderness should be such that:

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

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

For any other roof shape:

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

where h_r is the height of the apex above the top of the columns.

No limit need be placed on L_b/D when $\Omega < 1$ and the other symbols are as defined in Section 6.4.1.

6.5 Serviceability check – deflection

Deflections under service loading can govern the design of portal frames[28]. Generally, codes do not give specific limits for portal frame deflections. The responsibility for selecting suitable limits rests with the designer. Deflections should not impair the strength or efficiency of the structure or its components, nor cause damage to cladding and finishes.

It is recommended that deflections due to service loading should be calculated by computer analysis. The stiffness of a nominally pinned base may be taken as 20% of the column stiffness for the purpose of calculating deflections. In this case the base and foundations need not be designed for the resulting moment.

6.6 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 base connections. 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. 11

b) Calculate $H = M_p$ (column)/h_1

c) Take moments about the rafter hinge position giving:

$$M_p\,(\text{rafter}) = \frac{w'Lx}{2} - Hh_2 - \frac{w'x^2}{2}$$

d) Calculate r/L and L/h and then determine x from Fig. 12

e) Calculate w' from c)

f) Redesign the frame if the load capacity w' is less than the total factored load on the frame (per m).

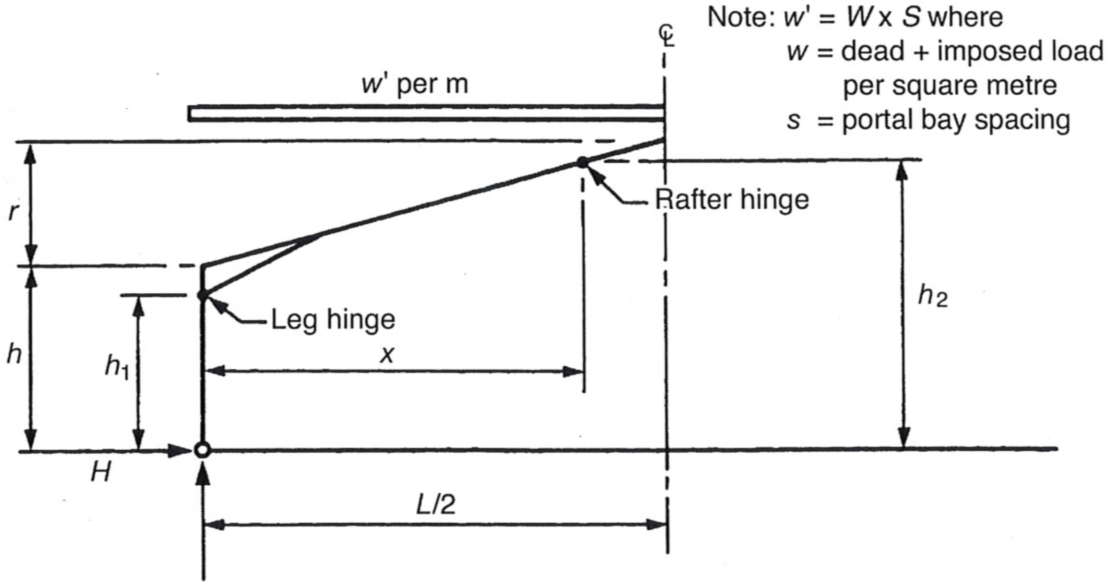


Fig. 11 Vertically loaded pinned-base portal frame

6.7 Stability checks

The following stability checks should be carried out:

- restraint of plastic hinges
- stability of rafter
- stability of haunch
- stability of column.

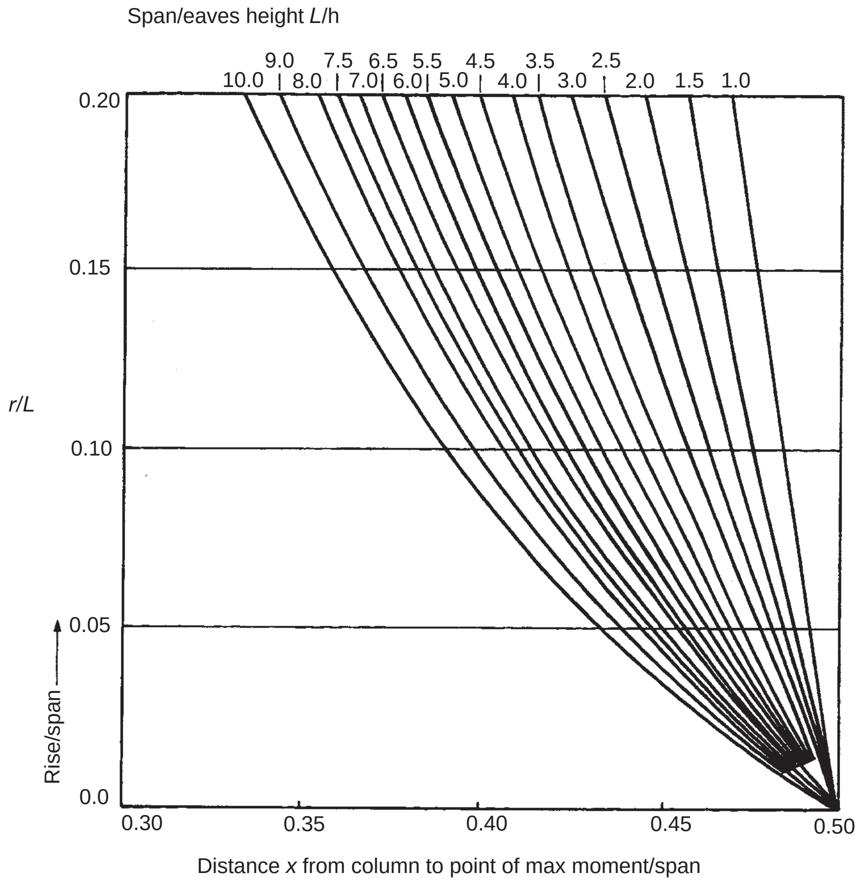


Fig. 12 Distance x from column to point of maximum moment/span

6.7.1 Restraint of plastic hinges

a) A restraint should be provided to both flanges at each plastic hinge location, designed to resist a force equal to 2.5% of the force in the compression flange. If placing the restraint directly at the hinge position 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_u in mm from the hinge restraint to the next adjacent restraint should not exceed

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

where
f_c is the average compression stress due to the axial load (N/mm^2)
p_y is the design strength (N/mm^2) from Table 2
r_y is the radius of gyration (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_u 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_s calculated as for the stability of haunch (see Section 6.7.3).

6.7.2 Rafter stability

The rafter should be checked to see that stability is maintained in all load cases. Under gravity loading, the following checks should be made:

- at the plastic hinge location near the ridge, both flanges should be laterally restrained in order to provide torsional restraint
- a purlin or other restraint is needed on the compression flange at a distance L_u calculated from the plastic hinge restraint formula given in Section 6.7.1
- further restraints to the top flange are required so that the rafter satisfies the requirements for beams without full lateral restraint
- in areas where there is compression on the bottom flange the procedure given for haunches in Section 6.7.3 should be applied using constants applicable to haunch/depth of rafter = 1.

6.7.3 Stability of haunch

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

a) the rafter is a UB section

b) the haunch flange is not smaller that the rafter flange

c) the depth of the haunch D_h is not greater than twice the depth of the rafter allowing for its slope, $2D_s$ see Fig. 9

d) the buckling resistance of the segments between the tension flange restraints is satisfactory when treated as though the flange is a compression flange and checked in accordance with the second equation of Section 10.5.2 using an effective length L_E equal to the spacing of the tension flange restraints.

L_s may conservatively be taken as:

$$L_s = \frac{620 r_y}{K_1 \sqrt{72 - \left(\frac{100}{x}\right)^2}} \text{ for S275 steel}$$

$$L_s = \frac{645 r_y}{K_1 \sqrt{94 - \left(\frac{100}{x}\right)^2}} \text{ for S355 steel}$$

where r_y is the minimum radius of gyration of the rafter section

x is the torsional index of the rafter section

K_1 has the following values:

for an unhaunched segment	$K_1 = 1.00$
for a haunch with $D_h/D_s = 1.0$	$K_1 = 1.25$
for a haunch with $D_h/D_s = 2.0$	$K_1 = 1.40$
for a haunch generally	$K_1 = 1 + 0.25\,(D_h/D_s)^{2/3}$

D_h is the additional depth of the haunch, see Fig. 9

D_s is the depth of the rafter allowing for its slope, see Fig. 9

However a more economic result may be obtained if the more complex expressions in Annex G of BS 5950-1:2000[1] are employed.

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

6.7.4 Stability of column

Near the top of the column a restraint to both flanges should be provided at the location of the plastic hinge, together with a further restraint at a distance L_u below the position of the hinge restraint.

If the column is restrained on the tension flange as described in Section 6.7.3d) then the distance to the nearest restraint on the compression flange may be taken as L_s as calculated for the stability of the haunch.

The column should then be checked in accordance with the overall buckling check in Section 10.5 to see if a further compression flange restraint is required.

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

7.1 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 axes (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 stresses in the chord members may be ignored providing that the loads are applied at the node points

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 first step in determining the size of the members of the lattice girder or truss is to calculate the total factored load on the roof from Section 4.2. The forces in the members for all relevant load combinations can be obtained by any suitable means including computer analysis. A computer analysis will require an initial estimate of the section sizes.

The design of members subject to compression, tension or combined axial force and bending is given below.

Compression members

Calculate the effective lengths L_E and design the section by reference to Section 10.4.2 or by reference to the compressive resistance of members in the tables in the blue book[2] as appropriate.

Tension members

Design the section by reference to Section 11.4 or to the tension capacity tables in the blue book[2].

Members subject to axial force and bending

For the type of member described in b) above that is subject to both axial force and bending, the design should be carried out in accordance with Section 10.5.

Deflection

The deflection of flat or horizontal lattice girders and trusses should be checked to see that serviceability with particular reference to roof drainage is not impaired. Ponding of rainwater could produce additional onerous load effects and accelerate deterioration of the roof covering.

Cambering of the structure may be considered to ensure roof drainage occurs. The side sway deflection at the column heads should be checked to ensure that they are within satisfactory limits.

7.2 Columns for single-storey buildings braced in both directions

The design procedure given below assumes that the bracing system is sufficiently stiff and well dispersed that the structure can be considered as 'non-sway' in both directions, see clause 2.4.2.6 of BS 5950-1:2000[1].

Design procedure

a) Calculate the unfactored axial load, F, on the column from the roof, and from the side cladding.

b) Calculate the unfactored wind loading on the side walls and on the roof.

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

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

e) Calculate the maximum unfactored moments arising from wind on the columns from:

$$M = (\text{greater of } W_{W1} \text{ or } W_{W2}) \times h/8$$

where h is the height of the column from base to eaves.

f) Calculate the unfactored nominal moments on the columns arising from the imposed and dead load by assuming a nominal eccentricity as for multi-storey columns or by elastic analysis and add these to the unfactored wind moments.

g) Select a section and check the design of the column as for Case II in Section 10.5 for the combined effects using the load combinations and load factors given in Table 1 of the *Manual*.

8.1 Gable posts

Calculate the unfactored axial forces and unfactored wind moments on these posts. Select a section and check the design of the gable post as for Case II in Section 10.5 for the combined effects using the load combinations and load factors given in Table 1 of the *Manual*. It will be found that a useful guide to the depth of the section for cases where there is wind and only nominal vertical load is $h/36$, where h is the height of the post.

8.2 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 11.

8.3 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.

8.4 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 Section 13.2 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 out as described in Section 13 for multi-storey buildings.

9.1 Introduction

This Section offers advice on the general principles to be applied when preparing a scheme for a braced multi-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. This aim has also to embrace the principle of designing for safety expressed in Section 2.2.

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, easy to build and can be erected safely. 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.

9.2 Loads

Loads should be based on BS 648:1964[7], BS 6399-1:1996[6], -2:1997[9], -3:1988[8].

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	24kN/m^3
•	density of lightweight aggregate concrete	19kN/m^3

*These are minimum values; for storey heights greater than 3.5m refer to BS 6399-1:1996[6].

9.3 Material selection

For multi-storey construction in the UK, grade S355 steel may be used for beams acting compositely with the floors or where deflection does not govern the design; otherwise grade S275 steel should be used for beams. For columns, grade S355 steel should be considered where it is intended to reduce the sizes to a minimum. Grade 8.8 bolts should normally be used throughout, preferably using a single diameter bolt. In the case parts of structures subject to vibration preloaded bolts should be considered.

9.4 Structural form and framing

The method for 'simple construction' as defined in BS 5950-1:2000[1] 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 Section 9.8).
e) Allow for movement joints (see Section 2.5).
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.

9.5 Fire resistance

In the absence of specific information, choose a fire-resistance period of 1 hour for the superstructure and 2 hours for ground floor construction over a basement and the basement structure.

Fire protection material must be sufficiently robust to withstand local damage during the normal use of the structure. Local building regulations should be consulted for requirements, which may be more severe than those shown above.

More detailed guidance is given in:

- BS 5950-8:2003[14]
- *Fire protection for structural steel in buildings*[15]
- *Guidelines for the construction of fire resisting structural elements*[23]
- *Structural fire safety: A handbook for architects and engineers*[29].

9.6 Corrosion protection

For multi-storey buildings on non-polluted inland sites general guidance on systems for protection of steelwork in certain locations follows. For other environments and for more detailed advice, reference should be made to BS 5493:1977[30] and to publications from Corus, the British Constructional Steelwork Association, the Zinc Development Association and the Paintmakers Association. The general guidance is as follows.

Steelwork integral with external cladding, particularly where not readily accessible for inspection and maintenance

- concrete encasement, or
- an applied coating system to give very long life such as:
  - hot-dip galvanize to BS EN ISO 1461:1999[31] (85μm), or
  - blast clean SA2½, isocyanate pitch epoxy (450μm) (BS 5493:1977[30], system reference SK8)

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 (l50μm), (SK5), or
- blast clean SA2½, 2 pack zinc-rich epoxy (70μm), epoxy MIO (l25μm), (SL3)

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)

Internal steelwork, heated building with negligible corrosion risk

It is feasible to avoid treatment altogether in the right environment. It is common in high rise structures, where the steel is dry and fully shielded from the weather, not to provide any protection, other than that needed to give fire resistance or decoration, to the structure.

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:

- shop applied:
  – blast clean to SA2½, HB zinc phosphate (70μm)
- site applied:
  – manual clean C St 2, non-oxidizing 'grease' paint (l00μm), or
  – manual clean C St 2, HB pitch solution (l50μm).

9.7 Bracing

Choose the location and form of bracing in accordance with the recommendations in Sections 2.2.3 and 9.4a). Typical locations are shown on Figs. 13 and 14 for different shaped buildings.

The wind load or the notional horizontal forces on the structure, whichever are greater, should be assessed and distributed to the bracing bays resisting the horizontal forces in each direction in a realistic manner. For symmetrically placed bracing bays of approximately equal stiffness, the distribution can be in equal proportions.

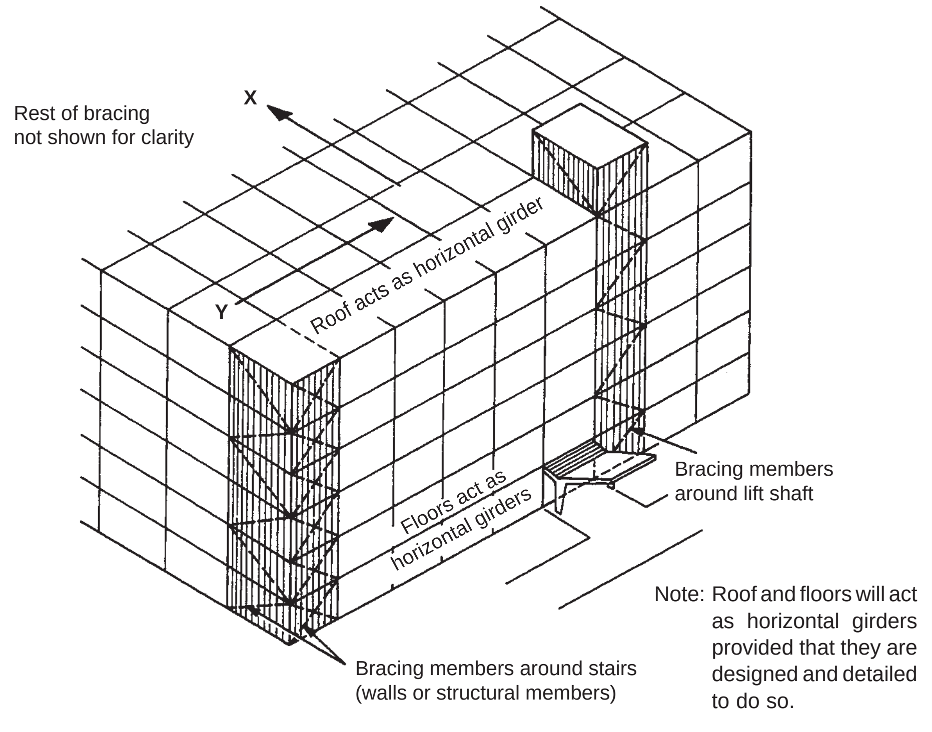


Fig. 13 Braced frame rectangular or square on plan

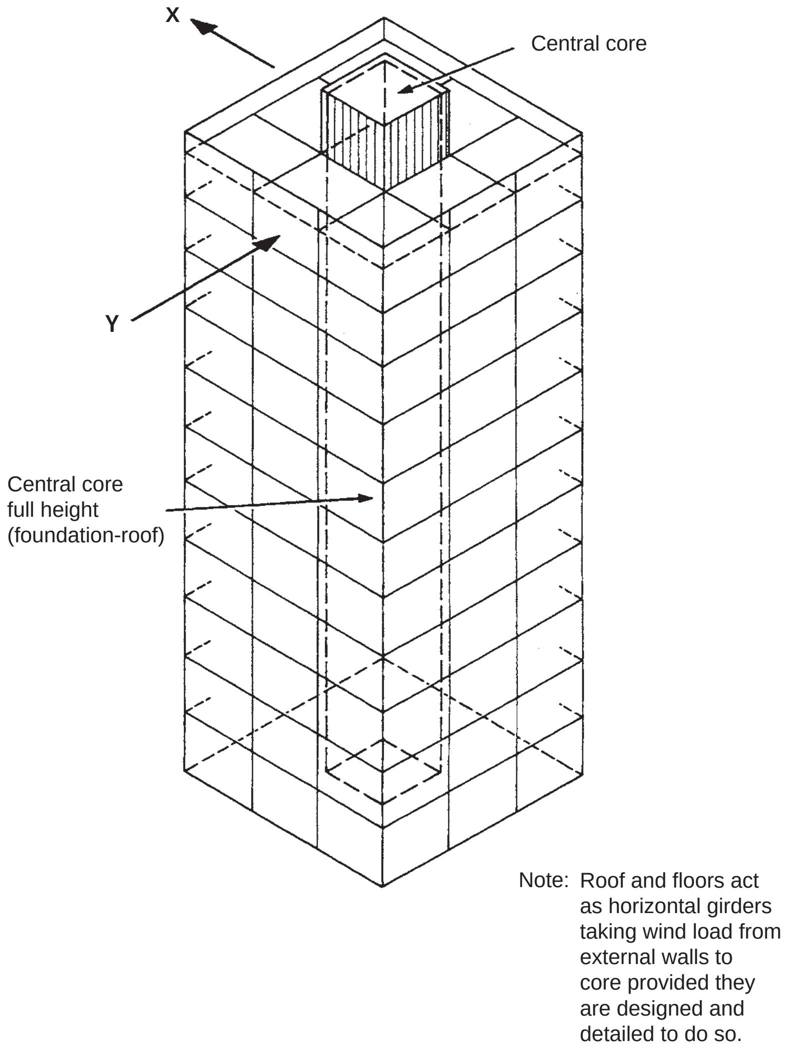


Fig. 14 Braced frame square on plan – central core

9.8 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 12 summarizes the salient features of the various types of flooring commonly used in the UK.

Table 12 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[a]	2.5 – 4	150 – 300	Medium	Poor	Poor	Domestic
In-situ concrete[b]	3 – 6	150 – 250	Medium	Very good	Very good	All categories but not often used for multi-storey steel construction as formwork and propping are required
Precast concrete[c]	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[d]	2.5 – 3.6 unpropped	110 – 150	Fast	Very good	Very good	All categories especially multi-storey commercial

Notes

a Timber floors should be designed to BS 5268-2:2002[32].

b In-situ concrete floors should be designed to BS 8110-1:1997[18] or to the IStructE/ICE *Manual for the design of reinforced concrete building structures*[33].

c Precast concrete floors should be designed to BS 8110-1:1997 and to the guides provided by the manufacturer of proprietary flooring systems.

d Profiled steel decking/composite floors should be designed to BS 5950-4:1994[34] and to the literature provided by the manufacturers of the proprietary metal decking systems.

10.1 Uncased columns

This Section describes the design of uncased columns for braced multi-storey 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 axes.

10.2 Determination of effective length of columns

For braced multi-storey 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 by the foundations. Fig. 15 illustrates typical joint and foundation restraint conditions; more information is contained in the *Lateral stability of steel beams and columns*[35].

It is most important to design the lateral restraints so that they have adequate stiffness and strength to inhibit movement of the restrained point in position or direction as appropriate. Each individual restraint should be capable of resisting not less than 1% of the maximum factored axial force in the column.

Positional restraints should be connected to a triangulated bracing system or shear diaphragm that must be capable of resisting the 1% restraint force. Where the bracing system serves more than one member, it should be designed for the sum of the restraint forces reduced by a factor k_r. The factor is given as:

$$k_r = \left(0.2 + \frac{1.0}{N_r}\right)^{0.5}$$

where N_r is the number of parallel members restrained.

The guidance given in this Section is appropriate to braced multi-storey structures that are classified by BS 5950-1:2000[1] as ‘non-sway’ (see Section 1.2 of the *Manual*).

10.3 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 Tables 11 and 12 of BS 5950-1:2000[1]. In this *Manual* slender sections are not considered for use in Case I.

Slender cross-sections have been identified (for axial compression only) in the blue book[2]. In order to assist the selection of suitable sections as columns for simple multi-storey construction it should be noted that all UCs and RSCs and most hollow sections, together with the Universal Beam sections shown in Table 13, which are not slender, could be chosen.

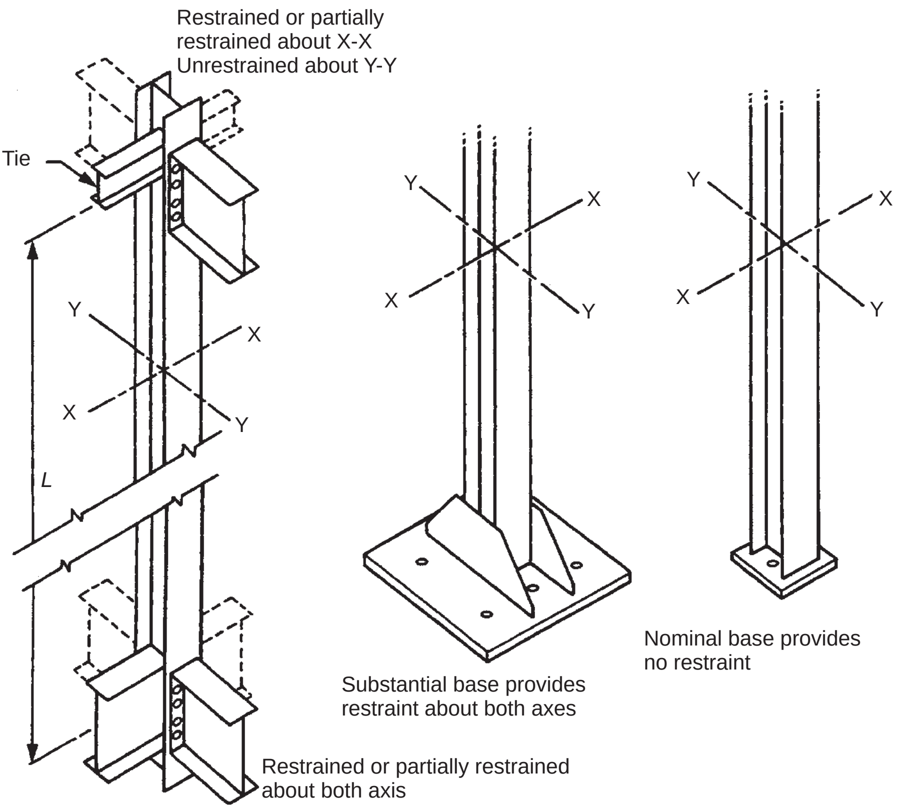
Table 13 Non-slender UB sections in compression

Grade S275	Grade S355
1016 x 305 × 487 and × 438 × 393	1016 × 305 × 487 and × 438 × 393
914 x 419 × 388	610 × 305 × 238
610 × 305 × 238 and × 179	356 × 171 × 67
533 × 210 × 122	305 × 165 × 54
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 and × 57	203 × 102 × 23
305 × 165 × 54 and × 46	178 × 102 × 19
305 × 127 × 48 and × 42 × 37	152 × 89 × 16
254 × 146 × 43 and × 37 × 31	152 × 76 × 13
254 × 102 × 28 and × 25 × 22	
203 × 133 × 30 and × 25	
203 × 102 × 23	
178 × 102 × 19	
152 × 89 × 16	
152 × 76 × 13	

10.4 Case I Columns braced in both directions – simple construction

For simple multi-storey 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 axes
e) nominal moments may be proportioned between the length above and below the beam connection according to the stiffness I/L of each length, except that when the ratio of the stiffness does not exceed 1.5 the moment may optionally 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) all equivalent uniform moment factors m should be taken as unity
h) the slenderness λ of the column should be limited to a reasonable value to allow practical detailing, for general robustness, for handling and erection. In the absence of better information, a limit of 180 may be applied.



Note: The restraint will only be fully effective if there is a substantial connection between the restraining beam and the column. This, ideally, should be by means of a plate welded to both flanges of the beam with adequate bolts connecting the members or substantial flange cleats to both flanges.

Fig. 15 Joint and foundation restraint conditions

Notes to simple construction method:

- the nominal moments as calculated in Section 10.4.1d) are the minimum moments to be used for column design
- 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 Section 10.5.

10.4.1 Design procedure

a) Calculate the factored beam reactions = 1.6 x imposed load + 1.4 x 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_c on the column being considered. In the preliminary design no allowance should be made for reduction in imposed load in accordance with BS 6399-1:1996[6], see Section 9.2.

c) Choose a trial section for the lowest column length from the following guide:
- 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 13. In that case the column design procedure described in Section 10.4.2 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 Section 10.4e).

f) Choose a section and grade of steel, for the lowest column length, such that the following equation is satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y Z_y} \leqslant 1$$

where

where	F_c	is factored axial load on the column
	P_c	is the compression resistance $p_c\,A_g$
	p_c	is the compressive strength
	A_g	is the gross cross-sectional area
	M_x	is the maximum nominal moment about the major axis
	M_y	is the maximum 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 effective length of the column on the y-y axis is 0.85 x the storey height the values of the compression resistance $A_g\,p_c$, the buckling resistance moment M_{bs} and the minor axis moment capacity $p_y Z_y$ may be obtained from the Tables in Appendix C for UC sections in S355 grade steel.

h) When these 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.

Column splices will normally be provided every two or three storeys up the building to facilitate transport, handling and erection – the length of column between splices usually being termed a ‘lift’. Advantage can be taken of these splices by reducing the column size in the upper storeys. Each change of section size will have to be checked using the procedure given above. Only the bottom storey of each ‘lift’ needs to be checked providing there are no significant differences in storey height or nominal moments at the higher levels in the same ‘lift’. Where it is economical to do so, a single section size can be used for the full height of the building.

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

As an alternative procedure to that described in Section 10.4.1g) the compressive resistance P_c of a column may be obtained from:

$$P_c = A_g p_c$$

where A_g is the gross sectional area of the trial section
p_c is the compressive strength

The alternative procedure is as follows.

a) Choose a trial section avoiding slender 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. UB sections acting as columns will normally be heavier than UC sections, unless there is a dominant moment effect.

b) Calculate the slenderness λ by dividing the effective length L_E obtained as in Section 10.2 by the radius of gyration of the chosen section about the relevant axis.

c) Determine p_c from Appendix D according to the type of section, axis of buckling, slenderness and the appropriate design strength p_y; for intermediate values of λ interpolation may be used.

d) To interpret these tables it should be noted that the definitions of I- and H-sections are as follows:

e) I-sections have a central web and two equal flanges, with an overall depth greater than 1.2 times the width of the flanges

f) 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.

g) Calculate the compressive resistance P_c from $A_g\,p_c$.

The scope of Appendix D includes the design of rolled I- or H-sections with welded flange plates using the appropriate plate type as shown in Fig. 16.

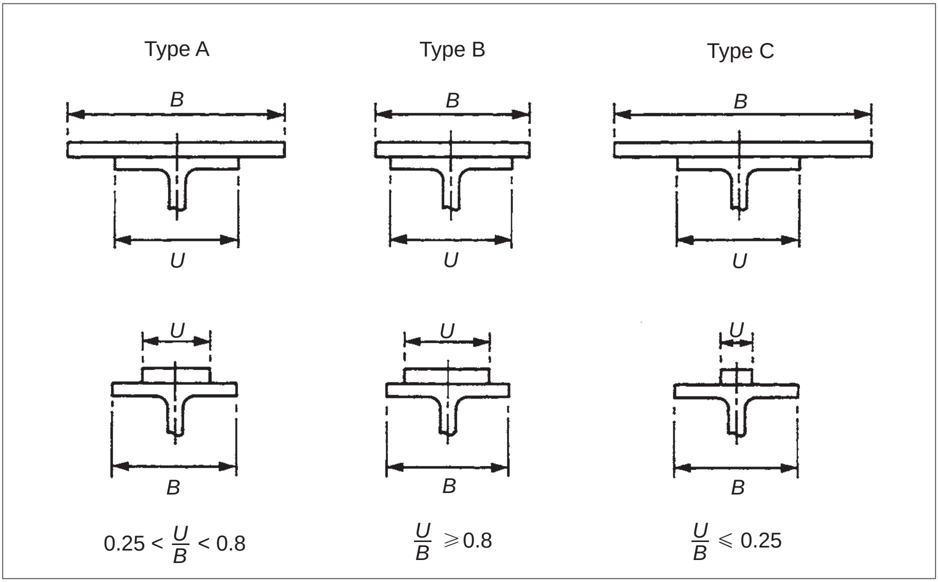


Fig. 16 Plate types for compound sections

10.4.3 Procedure for calculation of M_{bs} the buckling resistance moment for columns for simple construction

For circular or square hollow sections and for certain rectangular hollow sections (see BS 5950-1:2000[1]), the buckling resistance moment M_{bs} should be taken as equal to the moment capacity M_c of the cross-section. For semi-compact and slender sections, reference should be made to BS 5950-1:2000. For plastic or compact sections the buckling resistance moment M_{bs} is obtained from:

$$M_{bs} = p_b S_x$$

where S_x is the plastic modulus of the section about the major axis
p_b is the bending strength using an equivalent slenderness (except for rectangular hollows sections) of:

$$\lambda_{LT} \ \frac{0.5L}{r_y}$$

where L is the distance between levels at which both axes are restrained in position
r_y is the radius of gyration about the minor axis

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

Design procedure

a) Calculate the factored axial load F_c on the column at the level being considered = 1.6 x imposed load + 1.4 x dead load. It may also be necessary to calculate the axial load using different load factors for different load combinations. It is also essential to ensure that the critical load/moment combination has been determined, as this may not occur with the maximum axial load or the maximum moment. Pattern loading should be considered, see BS 5950-1:2000[1] clause 5.1.2.

b) Calculate the factored moments M_x and M_y on the major and minor axis, respectively, using the load combinations from a).

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 14 and m_{LT} from Table 6.

d) Choose a trial section avoiding slender 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.

10.5.1 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:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leqslant 1$$

The procedure to be followed is:

a) Having avoided the selection of slender sections as in Section 10.5d), determine the design strength p_y from Table 2 according to the grade of steel and the flange thickness.

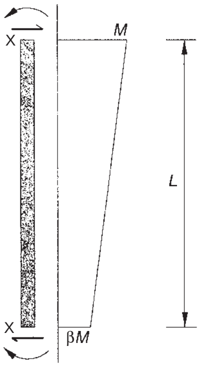
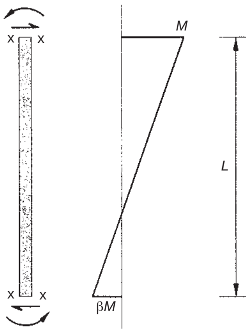
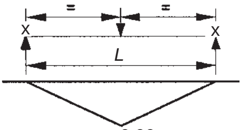
b) Calculate $\dfrac{F_c}{A_g p_y}$

c) Obtain the moment capacities M_{cx} and M_{cy} in the absence of axial load from the blue book[2] about the major and minor axis, respectively, and then calculate

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

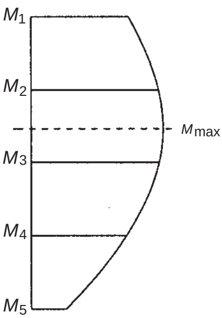
d) Finally, check that $\dfrac{F_c}{A_g p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} \leqslant 1$

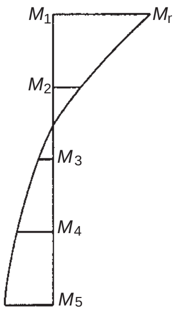
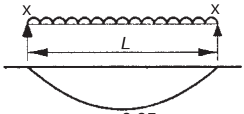
Table 14 Equivalent uniform moment factor *m* for flexural buckling

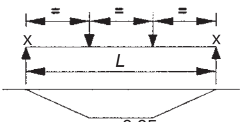
<table>
<tr><th colspan="2">Segments with end moments only (values of m from the formula for the general case)</th><th>β</th><th>m</th></tr>
<tr><td>β positive

</td><td>β negative

</td><td>1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0</td><td>1.00
0.96
0.92
0.88
0.84
0.80
0.76
0.72
0.68
0.64
0.60
0.58
0.56
0.54
0.52
0.50
0.48
0.46
0.44
0.42
0.40</td></tr>
<tr><th colspan="4">Segments between intermediate lateral restraints</th></tr>
<tr><th>Specific cases</th><th colspan="3">General case</th></tr>
<tr><td>

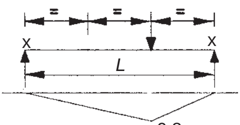
m = 0.90</td><td colspan="3" rowspan="3">

$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{\max}} \text{ but } m \geqslant \frac{0.8M_{2\text{-}4}}{M_{\max}}$$




</td></tr>
<tr><td>

m = 0.95</td></tr>
<tr><td>

m = 0.95</td></tr>
<tr><td>

m = 0.8</td><td colspan="3">The moments M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length.
If M_2, M_3 and M_4 all lie on the same side of the axis, their values are all taken as positive. If they lie on both sides of the axis, the side leading to the larger value of m is taken as the positive side.
The values of M_{max} and M_{2-4} are always taken as positive: where M_{max} is the maximum moment in the segment and M_{2-4} is the maximum moment in the central half of the segment.</td></tr>
</table>

10.5.2 Overall buckling check

This should be carried out by checking that:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leqslant 1 \text{ and } \frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leqslant 1$$

where F_c is the total compressive force due to axial load at the level being considered
P_c is the smaller of P_{cx} or P_{cy}
P_{cx} is the compression resistance, considering buckling about the major axis only
P_{cy} is the compression resistance, considering buckling about the minor axis only
A_g is the gross cross sectional area
M_b is the buckling resistance moment M_b, obtained as in Section 3, which should not be taken as greater than $p_y S_x$ for class 1 and 2 sections
M_x is the maximum moment about the major axis in the length governing P_{cx}
M_y is the maximum moment about the minor axis in the length governing P_{cy}
M_{LT} is the maximum moment about the major axis in the length governing M_b
p_y is the design strength obtained from Table 2
m_{LT} is the equivalent uniform moment factor for lateral torsional buckling from Table 6 for the pattern of major axis moments over the length governing M_b
m_x is the equivalent uniform moment factor for major axis flexural buckling from Table 14 for the pattern of major axis moments over the length governing P_{cx}
m_y is the equivalent uniform moment factor for minor axis flexural buckling from Table 14 for the pattern of minor axis moments over the length governing P_{cy}
Z_x is the elastic section modulus about the major axis
Z_y is the elastic section modulus about the minor axis

10.6 Cased columns

10.6.1 Introduction

This Section 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 Section 3.4.2 for cased beams.

10.6.2 Design procedure

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

a) For local capacity:

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

b) For overall buckling resistance:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leqslant 1 \text{ and } \frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leqslant 1$$

where F_c, M_b, M_{cx} and M_{cy} are calculated for the uncased section,
M_x, M_y, m_{LT}, m_x and m_y are as defined in Section 10.5.2

P_c is the compression resistance considering buckling about both axes given by

$$\left(A_g + \frac{0.45 A_c f_{cu}}{p_y}\right) p_c \text{ but } P_c \leqslant P_{cs}$$

where P_{cs} is the short strut capacity of the cased section given by:

$$\left(A_g + \frac{0.25 A_c f_{cu}}{p_y}\right) p_y$$

P_{cy} is the compression resistance calculated in the same way as P_c but considering buckling about the minor axis only

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 $\leqslant$ 40N/mm^2

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

p_c is the compressive strength of the steel member for buckling about the relevant axis based on the slenderness λ given by:

for buckling about the major axis: L_E/r_x

for buckling abut the minor axis: $L_E/(0.2b_c)$

but $\geqslant L_E/[0.2(B+150)]$ and

$\leqslant L_E/r_y$ for the uncased section

where L_E is the effective length for buckling about the relevant axis

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

r_y is the radius of gyration of the steel section alone about its minor axis

b_c is the minimum width of solid casing within the depth of the steel section

B is the overall width of the steel flange

11.1 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.

The slenderness λ obtained by dividing the effective length L_E by the radius of gyration about the relevant axis should be limited to a reasonable value to allow practical detailing, for general robustness and for handling and erection. Whilst not a requirement of BS 5950-1:2000[1], in the absence of better information the following limits may be applied:

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

11.2 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 Section 10.4.2. For discontinuous struts composed of angles, channels or T-sections, the eccentricities arising from the connections may be ignored, and members shown in Table 15 may be designed as axially loaded only.

The slenderness should be obtained from Table 15, 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 15.

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.

11.3 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 Section 10.5. For sections other than angles and T-sections the section should be chosen as indicated in Section 10.3 in order to avoid the use of slender elements. For angles and T-sections, the section should be chosen such that:

- for angles, b/t and $d/t \leqslant 15\varepsilon$ and $(b+d)/t \leqslant 24\varepsilon$
- for T-sections, $b/T \leqslant 15\varepsilon$ and $D/t \leqslant 18\varepsilon$

where $\varepsilon = \left[\dfrac{275}{p_y}\right]^{0.5}$

11.4 Bracing members in tension only

The tension capacity of a member should be calculated from

$$P_t = A_e\, p_y$$

where A_e is the effective area of the section but not more than 1.2 x the total net area A_n

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

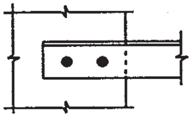
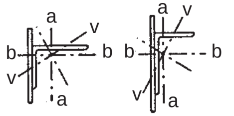
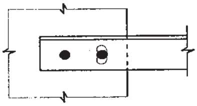
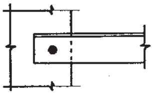
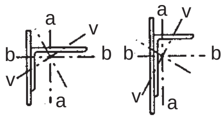
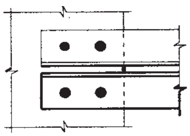
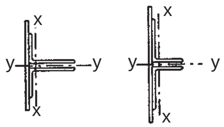
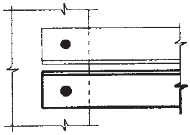
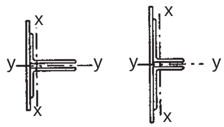
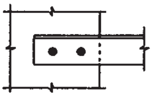
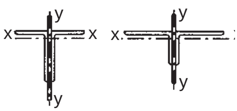
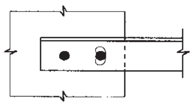
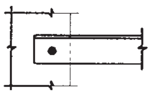
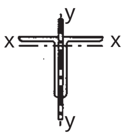
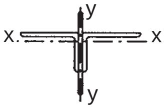
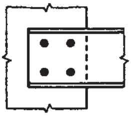
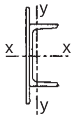
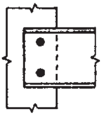
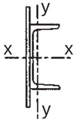
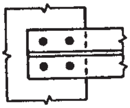
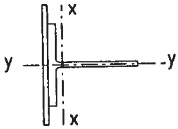
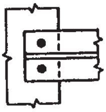
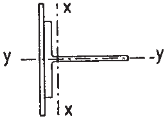
Table 15 Slenderness for discontinuous angle, channel and T-section struts		
Connection	**Sections and axes**	**Slenderness ratios** [a, b, c]
		v-v axis: $0.85L_v/r_v$ but $\geqslant 0.7L_v/r_v + 15$ a-a axis: $1.0L_a/r_a$ but $\geqslant 0.7L_a/_{ra} + 30$ b-b axis: $0.85L_b/r_b$ but $\geqslant 0.7L_b/r_b + 30$
 (kidney-shaped slot) 		v-v axis: $1.0L_v/r_v$ but $\geqslant 0.7L_v/r_v + 15$ a-a axis: $1.0L_a/r_a$ but $\geqslant 0.7L_a/r_a + 30$ b-b axis: $1.0L_b/r_b$ but $\geqslant 0.7L_b/r_b + 30$ (see note d)
		x-x axis: $1.0L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $[(0.85L_y/r_y)2 + \lambda_c^2]^{0.5}$ but $\geqslant 1.4\lambda_c$ (see notes e and f)
		x-x axis: $1.0L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)2 + \lambda_c^2]^{0.5}$ but $\geqslant 1.4\lambda_c$ (see notes d, e and f)
		x-x axis: $0.85L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)^2 + \lambda_c^2]^{0.5}$ but $\geqslant 1.4\lambda_c$ (see notes e and f)
 (kidney-shaped slot) 	 	x-x axis: $10L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)2 + \lambda_c^2]^{0.5}$ but $\geqslant 1.4\lambda_c$ (see notes d, e and f)

Table 15 Slenderness for discontinuous angle, channel and T-section struts

Connection	Sections and axes	Slenderness ratios [a, b, c]
		x-x axis: $0.85L_x/r_x$ y-y axis: $1.0L_y/r_y$ but $\geqslant 0.7L_y/r_y + 30$
		x-x axis: $1.0L_x/r_x$ y-y axis: $1.0L_y/r_y$ but $\geqslant 0.7L_y/r_y + 30$
		x-x axis: $1.0L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $0.85L_y/r_y$
		x-x axis: $1.0L_x/r_x$ but $\geqslant 0.7L_x/r_x + 30$ y-y axis: $1.0L_y/r_y$

Notes

a 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.

b Intermediate restraints reduce the value of L for buckling about the relevant axes. For single angle members, L_v is taken between lateral restraints, perpendicular to either a-a or b-b.

c For single-angle struts with lateral restraints to its two legs, alternately, the slenderness for buckling about each axis should be increased by 20%.

d 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 5950-1:2000[1] clause 4.7.10.2b or 4.7.10.3d.

e Double angles are either battened, see BS 5950-1:2000[1] clause 4.7.12 or interconnected back-to back, see BS 5950-1:2000[1] clause 4.7.13. Battens or interconnecting fasteners are also needed at the ends of members.

f $\lambda_c = L_v/r_v$ with L_v measured between interconnecting fasteners for back-to-back struts or between end welds or end fasteners of adjacent battens for battened angle struts.

11.4.1 Angles, channels and T-sections

In simple tension members composed of angles, channels or 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:

$P_t = p_y (A_e - 0.5a_2)$ for bolted connections and
$P_t = p_y (A_g - 0.3a_2)$ for welded connections

where	P_t	is the tension capacity of the member
	p_y	is the design strength
	A_e	is the effective area of the whole section = a_e of the connected element plus area of the outstand leg
	a_2	is the area of the outstand elements = $A_g - a_1$ (Fig. 17)
	A_g	is the gross area of the section
	a_1	is the gross area of the connected element, taken as the product of the thickness and the overall element width for an angle, the overall depth for a channel or the flange width for a T-section

The effective net area a_e of an element is taken as K_e x net area a_n but not greater than the gross area a_g.
K_e is the effective area coefficient given by:

- $K_e = 1.2$ for grade S275
- $K_e = 1.1$ for grade S355
- for other grades see BS 5950-1:2000[1].

b) Double angle, channel and T-section ties connected to both sides of a gusset or section.
Where two angles are connected through one leg, two channels are connected through the web or two T-sections are connected through the flange to both sides of a gusset or section and:

- are held apart and longitudinally parallel by battens or solid packing pieces
- are interconnected using bolts or welds in at least two locations within their length (a recommendation for the spacing of interconnections is illustrated in Fig. 18 in which λ is the minimum value for the individual components)
- are provided with the outermost interconnections within a distance from each end of ten times the smaller leg length for angles or the smaller overall dimension for channels and T-sections, the tension capacity of each component should be taken as:
  for bolted connections: $P_t = p_y (A_g - 0.25a_2)$
  for welded connections: $P_t = p_y (A_g - 0.15a_2)$
  where P_t, p_y, A_e, A_g and a_2 are as described previously.

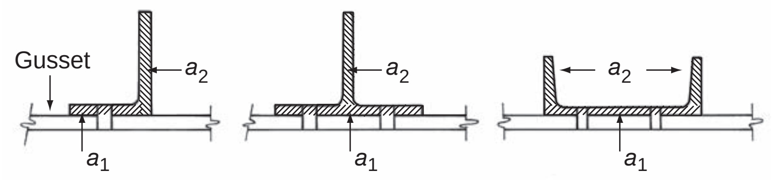


Fig. 17 Illustrations for areas a_1 and a_2

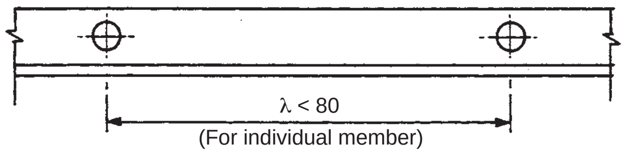


Fig. 18 Slenderness of individual member

c) Single angles connected through both legs, single channels connected through both flanges, single T-sections connected through the stem only or through the stem and the flange, and internal bays of continuous ties.
For these types of connection and member the effective area should be taken as the sum of the effective net areas of each element of the section.

d) For double angle, channel and T-section ties that are connected to one side of a gusset or section or are not interconnected as required in 11.4.1b), each component should be designed in accordance with 11.4.1a).

11.4.2 Other sections

The tension capacity of sections, other than those given in Section 11.4.1, may be found from:

$$P_t = A_e p_y$$

where A_e the effective area, is taken as the sum of the effective net areas, a_e, of all the elements of the section at the connection. The effective net area of each element should be determined as in Section 11.4.1a). In no case should the effective area A_e be taken as greater than the gross area A_g of the section nor greater than 1.2 times the total net area A_n of the section.

11.5 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:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} < 1$$

where F_t is the factored axial load in member

P_t is the tension capacity, derived as shown previously = $A_e\, p_y$

A_e is the effective area as defined in Section 11.4

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[2]

M_{cy} is the moment capacity about the minor axis in the absence of axial load obtained, from the blue book[2]

These members should also be checked for bending strength only as given in Section 3.

12.1 Introduction

Multi-storey construction that has been framed in accordance with the recommendations given in Section 9.4 and designed in accordance with the rest of the *Manual*, should produce a robust construction providing the requirements for disproportionate collapse are met and subject to the connections also being designed in accordance with the *Manual*.

Changes to the Building Regulations[36] and amendments to BS 5950-1:2000[1] (AMD 17137) mean disproportionate collapse must be considered for all buildings. The requirements deemed necessary to take account of disproportionate collapse depend on the classification of the building. There are four building classes:

- Class 1 includes houses not exceeding four storeys; agricultural buildings; buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height.
- Class 2A includes five storey single occupancy houses; hotels not exceeding four storeys; flats, apartments and other residential buildings not exceeding four storeys; offices not exceeding four storeys; industrial buildings not exceeding 3 storeys; retailing premises not exceeding three storeys of less than 2,000m^2 floor area in each storey; single storey educational buildings; all buildings not exceeding two storeys to which members of the public are admitted and which contain floor areas not exceeding 2,000m^2 floor area at each storey.
- Class 2B includes hotels, flats, apartments and other residential buildings greater than four storeys but not exceeding 15 storeys; educational buildings greater than one storey but not exceeding 15 storeys; retailing premises greater than three storeys but not exceeding 15 storeys; hospitals not exceeding three storeys; offices greater than four storeys but not exceeding 15 storeys; all buildings to which members of the public are admitted which contain floor areas exceeding 2,000m^2 but less than 5,000m^2 at each storey; car parking not exceeding six storeys.
- Class 3 includes all buildings defined above as Class 2A and 2B that exceed the limits on area or number of storeys; grandstands accommodating more than 5,000 spectators; buildings containing hazardous substances or processes.

12.2 Class 1 and Class 2A Buildings

The recommended requirements for avoidance of disproportionate collapse of Class 1 and Class 2A buildings are as follows.

- Columns should be tied in two directions, approximately at right angles, at each principal floor level.
- All ties (as close as practicable to the edges of the floors and roof and along each column line) and their end connections should be capable of resisting a factored tensile load of at least 75kN.
- Horizontal ties should also be provided at roof level, except where steelwork only supports cladding that weighs not more than 0.7kN/m^2 and that carries only imposed roof loads and wind loads.

Fig. 19 shows which members need to be designed as ties for Class 1 and Class 2A buildings. In practice, the required tying capacity of 75kN is achieved by any reasonable member and connection.

12.3 Class 2B Buildings

For Class 2B buildings, the recommended requirements for avoidance of disproportionate collapse of Class 1 and Class 2A buildings should be followed along with the following design requirements.

There are three possible methods that can be adopted for designing to avoid disproportionate collapse:

- provision of tying
- notional removal of elements
- key element design.

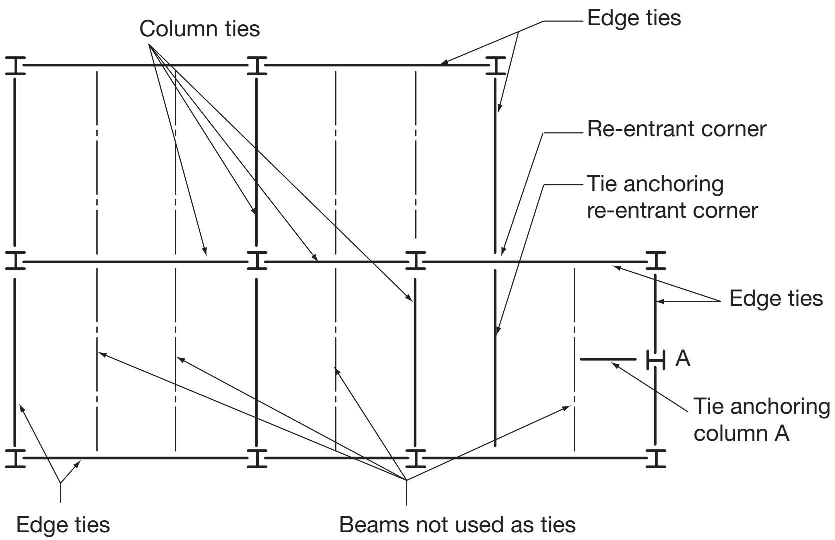


Fig. 19 Tying of columns in Class 1 and Class 2A buildings (BS 5950-1 2000[1] Figure 1)

For the provision of tying method, the requirements for disproportionate collapse may be assumed to be satisfied provided that the following five conditions a) to e) are met.

a) Beams or ties should be provided in two directions approximately at right angles at each floor and roof level. These ties and their end connections should be checked for the following tensile forces, which need not be considered as additive to other loads:

Internal ties: $0.5\ (1.4g_k + 1.6q_k)\ s_t\ L\ n$

Edge ties: $0.25\ (1.4g_k + 1.6q_k)\ s_t\ L\ n$

where g_k is the total specified dead load per unit area

q_k is the total specified imposed load per unit area

s_t is the mean transverse spacing of the ties adjacent to the tie being checked

L is the span

n is a reduction factor relating to the number of storeys in the structure (n = 0 for 1 storey, n = 0.25 for 2 storeys, n = 0.5 for 3 storeys, n = 0.75 for 4 storeys and n = 1.0 for 5 or more storeys).

Alternatively the tie force may be taken as the larger of the two end reactions of the member under factored loads multiplied by n. In no case should the tie force be taken as less than 75kN.

For composite and in-situ floors the reinforcement in the floor construction, providing it is suitably designed and detailed, may be used to resist the tie forces (reference should be made to BS8110-1:1997[18]).

For the purposes of calculating the tying capacity for avoidance of disproportionate collapse, it may be assumed that substantial deformation of members and their connections is acceptable.

A typical arrangement of ties in a Class 2B building is shown in Fig. 20.

b) Columns nearest to the edges of the floors and roof should be anchored with horizontal ties, acting perpendicular to the edge with a tying capacity at least equal to the tie force defined in a) above and not less than 1% of the factored vertical load in the column at the level of the tie.

c) Column splices should be checked for a tensile force equal to the largest sum of the factored vertical dead and imposed load reactions from all the beams connected to the column at a single floor level located between that column splice and the next column splice below. All columns should be carried through at each beam to column connection.

d) The bracing system should be distributed throughout the building such that, in each of two directions approximately at right angles, no substantial portion of the building is connected at only one point to a bracing system.

e) Precast concrete or other heavy floor and roof units should be effectively anchored in the direction of their span either to each other over supports or directly to their supports in accordance with BS8110-1:1997[18].

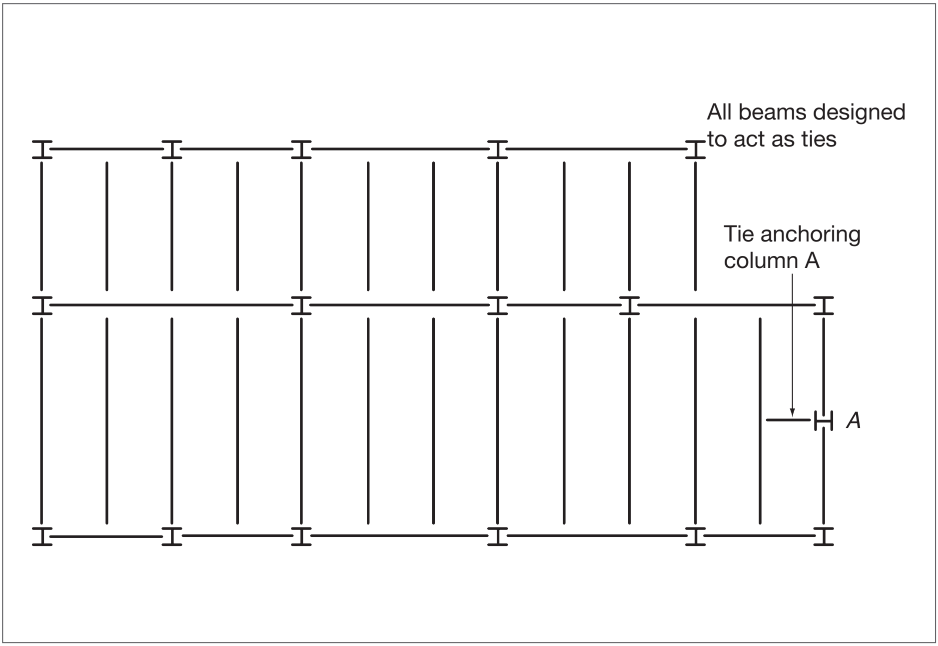


Fig. 20 Tying of columns in Class 2B buildings (BS 5950-1:2000[1] Figure 2)

If any of the conditions a) to e) is not met, the building should be designed to avoid disproportionate collapse using either the notional removal of elements method or key element design.

The notional removal of elements method should be applied.

- Where a), b) or c) is not met, the area of damage, following notional removal at each storey in turn of a single column or beam supporting a column, should be assessed.
- Where d) is not met, the area of damage, following notional removal at each storey in turn of a single element of the bracing system, should be assessed.

The area of damage should be limited to the lesser of 15% of the floor or roof area or 70 m^2 at the relevant level and at one immediately adjoining floor or roof level, either above or below it, as shown in Fig. 21.

When carrying out the check for the area of damage the following should be noted.

- 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.
- 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.
- 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.

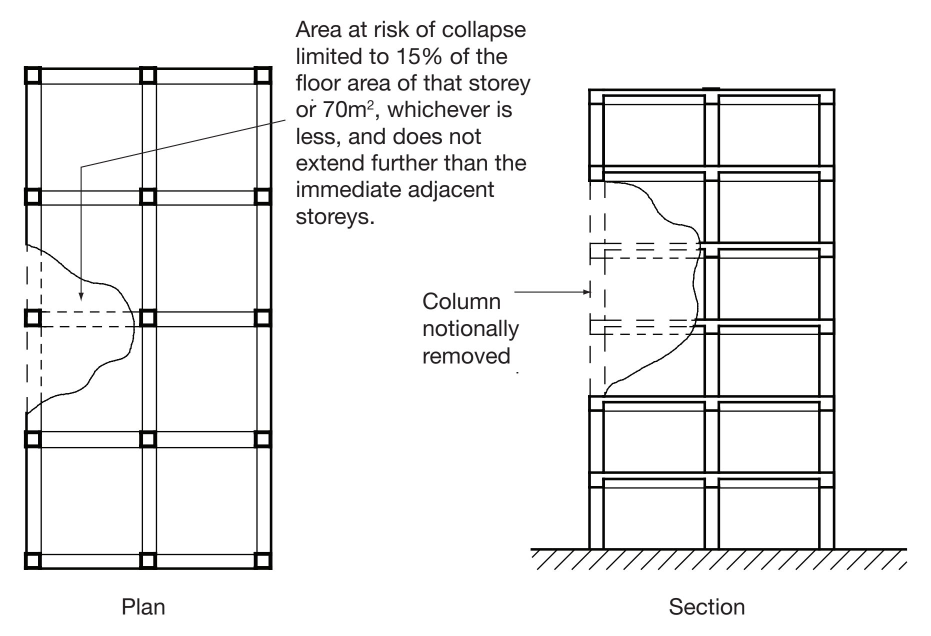


Fig. 21 Damage caused by notional removal of a column (Building Regulations[36] Approved Document A Diagram 25)

Where the notional removal of elements conditions are not satisfied the relevant member and any other member or component vital to the stability of the member under consideration should be designed as 'key elements'.

For key element design, the stipulated accidental load (taken as 34kN/m^2, see BS 6399-1:1996[6]) should be applied from all horizontal and vertical directions, one at a time, together with the reactions from any other member or component designed for the same accidental loading but limited to the ultimate strength of these components or their connections.

In conjunction with the accidental loads, the effects of ordinary loads should be considered using the loads and the same partial safety factors as used for the notional removal of elements method described previously.

12.4 Class 3 Buildings

For Class 3 buildings, the recommended requirements for avoidance of disproportionate collapse of Class 2B should be followed. In addition, a systematic risk assessment should be carried out to determine the normal and abnormal hazards that can reasonably be foreseen during the lifetime of the building. The structure should be designed to ensure that any collapse is not disproportionate to the cause.

Further guidance for designing steel framed buildings to avoid disproportionate collapse is provided in SCI publication *Guidance on meeting the Robustness Requirements in Approved Document A (2004 Edition)*[37].

13.1 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.

13.2 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.

13.3 Completion of design

Before the design of the structure can be finalised, 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 marshal 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 and any assumptions made during the analysis and design process
- amendments of drawings as a basis for final calculations
- design/checking iteration.

13.3.1 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. This may include amongst others the following:

Erection

Develop a proposed/assumed and safe method of erection that is inherent within the strength and stability (permanent and temporary) checks at design stage. Construction stage method/sequence of erection and temporary works (e.g. erection bracing) must be reviewed to ensure that these are consistent with and in no way compromise the assumptions made at design stage; otherwise modifications must be made to the design and/or construction assumptions. Reference should be made to the BCSA *Guide to steel erection*[38] series.

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. Variations in ground water level may need to be considered.

Holes and openings for services

Ensure holes and openings required are positioned and if necessary reinforced.

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.

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.

Movement joints

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

Performance criteria

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

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.

13.3.2 Preparation of design data list

The information obtained from the above checklist 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.

13.3.3 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.

13.3.4 Design/checking iteration

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 preceding sections, for example:

- 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 preceding sections.

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 connection behaviour should be consistent with the assumptions made in determining the forces around the building frame. 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 and their connections.

b) In columns, splice connections between rolled I- or H-sections may be assumed to be in direct bearing when the ends of the sections are cut square using a good quality saw in proper working order. It must be emphasised that direct bearing does not necessitate the machining or end milling of the column and that full contact over the whole column is not essential (see BS 5950- 2:2001[39], and the National Structural Steelwork Specification[40]). In other situations, the bolts, welds and splice plates should be designed to carry all the forces at the splice position.

c) In bolted moment connections, the simple force distribution shown in Fig. 22 may be used. More sophisticated models are included in the SCI/BCSA publication[41] for moment connection design.

d) As far as possible only one diameter and grade of bolts should be used on a project. Bolts should generally be of 8.8 grade and not less than 12mm 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 sherardized, 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 should be incorporated in connections subjected to vibration impact, or when slip of the connections, which could lead to increased or excessive deflection, is unacceptable.

It is particularly important to check the connection on site.

Critically loaded welded connections should be tested using non-destructive methods such as ultrasonic tests to BS EN 1714:1998[42] for butt welds and magnetic particle inspection to BS EN ISO 9934-1:2001[43] for fillet welds.

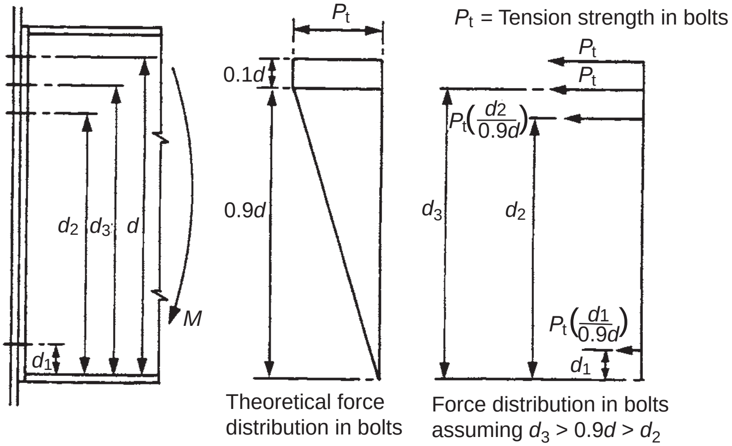


Fig. 22 Force distribution on bolts

14.2 Bolts

14.2.1 Spacing and edge distances

A summary of the requirements for standard clearance holes is given in Table 16. For oversize, slotted and kidney shaped holes, reference should be made to BS 5950-1: 2000[1].

Table 16 Bolt spacing and edge distances

Requirement	**Distance**
Minimum spacing	2.5d
Maximum spacing in unstiffened plate: in direction of stress in any environment exposed to corrosion in any direction	 14t 16$t \leqslant$ 200mm
Minimum edge and end distance: rolled, machine flame cut, sawn or planed edge or end sheared or hand flame cut edge or end	 1.25D 1.4D
Maximum edge distance: normal exposed	 11$t\varepsilon$ 40mm + 4t

Nomenclature
t is the thickness of the thinner part
d is the nominal bolt diameter
D is the hole diameter
ε is $\sqrt{\frac{275}{p_y}}$

Table 17 Strength checks for bolts

Required strength	Formula
Shear capacity	$P_s = p_s A_s$
Bearing capacity of bolt	$P_{bb} = d t_p p_{bb}$
Bearing capacity of ply	$P_{bs} = k_{bs} d t_p p_{bs} \leqslant 0.5 k_{bs} e t_p p_{bs}$
Large grips where the total thickness of the connected plies exceeds 5 x the nominal diameter d of the bolts – shear capacity	$P_s = p_s A_s \left(\frac{8d}{3d + T_g} \right)$
Long joints where distance between the first and last rows of bolts in the direction of the load exceeds 500mm – shear capacity	$P_s = p_s A_s \left(\frac{5500 - L_j}{5000} \right)$
Packing where the thickness of the packing in the shear plane is greater than $d/3$ the shear capacity should be taken as:	$P_s = p_s A_s \left(\frac{9d}{8d + 3t_{pa}} \right)$
The thickness of the packing should not be greater than:	$4d/3$
Tension capacity	$P_t = 0.8 p_t A_t$
Combined shear and tension	$\frac{F_s}{P_s} + \frac{F_t}{P_t} < 1.4$ but no part should be greater than 1.0

Nomenclature

p_s is the shear strength obtained from Table 18
p_{bb} is the bearing strength of the bolt obtained from Table 18
p_{bs} is the bearing strength of connected part obtained from Table 19
e is the end distance but not greater than $2d$
p_t is the tension strength of the bolt obtained from Table 18
A_s is the effective area for shear, taken as the tensile stress area. If not known then the area at the bottom of the threads
A_t is the tensile stress area
L_j is the lap length of the joint, taken as the distance between the first and last rows of bolts in one half of the joint, measured in the direction of load transfer (mm)
T_g is the thickness of the grip (mm)
d is the nominal bolt diameter
t_p is the thickness of the connected part or, if the bolts are counter sunk, the thickness of the part minus the depth of the counter sinking
t_{pa} is the total thickness of steel packing at a shear plane

Provided that the sizes of holes for non-preloaded bolts do not exceed the standard dimensions given in BS 5950-1: 2000[1] Table 33, the coefficient k_{bs} should be taken as follows:

- for bolts in standard clearance holes: $k_{bs} = 1.0$
- for bolts in oversize holes: $k_{bs} = 0.7$
- for bolts in short slotted holes: $k_{bs} = 0.7$
- for bolts in long slotted holes: $k_{bs} = 0.5$
- for bolts in kidney shaped slots: $k_{bs} = 0.5$

14.2.2 Strength checks

The strength of ordinary bolts to carry the forces should be checked using the formulae in Table 17.

It should be noted that normal bolts are frequently supplied with long thread lengths, leading in many cases to threads being present in the shear plane. Unless the designer is certain that this situation does not exist it should be assumed that there are threads present in the bolts at this point in the bolt and design should be in accordance with this assumption.

The tensile capacity in Table 17 is based on a simple method that requires the bolt cross-centre spacing to be less than 55% of the flange or plate width and the moment capacity of the connected part per unit length to be taken as $p_y t_p^2/6$ when designed assuming double curvature bending. In other cases or where the designer wishes to use a higher strength then BS 5950-1:2000[1] clause 6.3.4.3 should be consulted.

Table 18 Values of p_s, p_{bb}, p_t

	Bolt grade			Other grades of fasteners $U_b \leqslant 1000$ (N/mm²)
	Grade 4.6 (N/mm²)	Grade 8.8 (N/mm²)	Grade 10.9 (N/mm²)	
Shear strength, p_s	160	375	400	$0.4U_b$
Bearing strength, p_{bb}	460	1000	1300	$0.7(U_b + Y_b)$
Tension strength, p_t	240	560	700	$0.7U_b$ but $\leqslant Y_b$

Nomenclature

Y_b is the specified minimum yield strength of the fastener

U_b is the specified minimum ultimate tensile strength of the fastener

Table 19 Values of p_{bs}

	Steel to BS EN 10025:1993[16] or BS EN 10113:1993[17]		Other grades of steel $U_b \leqslant 1000$ (N/mm²)
	Grade S275 (N/mm²)	Grade S355 (N/mm²)	
p_{bs}	460	550	$0.67(U_s + Y_s)$

Nomenclature

p_{bs} is the bearing strength of the connected part

Y_s is the specified minimum yield strength of the steel

U_s is the specified minimum ultimate tensile strength

U_b is the specified minimum ultimate tensile strength of the fastener

Table 20 Minimum shank tension P_o (proof load) for general grade preloaded bolts	
Bolt diameter (mm)	**Proof load (kN)**
M12	49.4
M16	92.1
M20	144.0
M24	207.0
M30	286.0
Note a For preloaded bolts the formula in Table 21 should be used.	

Table 21 Strength check for preloaded bolts	
Required strength	**Formula**
Slip resistance (parallel shank bolt designed to be non-slip at service loads)	$P_{sL} = 1.1K_s\mu P_o$ but $\leqslant$ each of the following $P_{bg} = 1.5dt_p p_{bs}$ $P_{bg} = 0.5et_p p_{bs}$ $P_s = p_sA_s$ (see Note a)
Slip resistance (waisted shank bolts and bolts designed to be non-slip at factored loads)	$P_{sL} = 0.9K_s\mu P_o$
Combined shear and tension for connection designed to be non-slip at service loads	$F_s/P_{sL} + F_{tot}/(1.1\ P_o) \leqslant 1$ but F_{tot} p_tA_t (see Note b)
Combined shear and tension for connection designed to be non-slip at factored loads	$F_s/P_{sL} + F_{tot}/(0.9\ P_o) \leqslant 1$ (see Note b)

Nomenclature

P_o is the minimum shank tension obtained from Table 20
K_s is 1.0 for standard clearance holes
0.85 for oversize holes, for short slotted holes and for long slotted holes loaded perpendicular to the slot*
0.7 for long slotted holes loaded parallel to the slot*
(*For definition and use reference should be made to BS 5950-1:2000[1])
μ is the slip factor taken from Table 35 of BS 5950-1:2000[1]
F_s is the applied shear
F_{tot} is the total applied tension in the bolt allowing for any prying
p_t is the tension strength of the bolt which, for general grade HSFG bolts[44] is taken as:
590 N/mm^2 for $\lambda \leqslant$ M24
515 N/mm^2 for $\lambda \geqslant$ M27
A_t is the tensile stress area
All other nomenclature is defined in Table 17

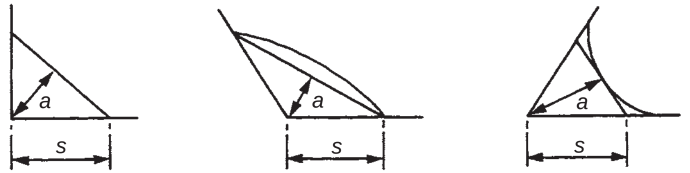
Notes

a The shear capacity should be calculated as for ordinary bolts taking account of large grip lengths, long joints and thickness of packing, see Table 17.
b In the expression for non-slip at service loads $P_{sL} = 1.1K_s\mu P_o$ whilst in that for non-slip at factored loads $P_{sL} = 0.9K_s\mu P_o$
c The capacities of bolts are also tabulated in the blue book[2].

14.3 Welds

14.3.1 Fillet welds

Fillet welds are designed using an effective throat thickness *a* as shown in Fig. 23 Special measures should be taken when the fusion faces form angles greater than 120° or less than 60°. 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 22. A load bearing fillet weld should be longer than 4 x the leg length, and longer than 40mm.



s = leg length = specified nominal weld size
a = throat thickness

Fig. 23 Fillet weld:effective throat thickness

Table 22 Design strength pw of fillet welds (N/mm²)

Steel grade	Electrode classification 35	42	50	For other types of electrode and/or other steel grades:
S275	220	(220)[a]	(220)[a]	$p_w = 0.5\ U_e$ but $p_w \leqslant 0.55U_s$ where
S355	(220)[b]	250	(250)[a]	U_e is the minimum tensile strength of the electrode as specified in the relevant product standard U_s is the specified minimum tensile strength of the parent material

Notes
a Over matching electrodes.
b Under matching electrodes. Not to be used for partial penetration butt welds.

The strength of fillet welds may be taken as that of the parent metal in the following situations:

- the welds have equal legs and are symmetrically disposed
- the grade of the parent metal is S275
- the welds connect two elements that are at right angles to each other
- the welds are subject to direct tension or compression only.

For other situations, the vectorial sum of the design stresses may be calculated and limited to the values given in Table 22. Alternatively, the directional method, given in clause 6.8.7.3 of BS 5950-1:2000[1], may be used.

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 the member. For partial penetration welds, it should be taken as the minimum depth of weld penetration. 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[2].

15 Typical connections

In this *Manual* a number of typical connections for braced multi-storey 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 may be made to publications of the Steel Construction Institute and the British Constructional Steelwork Association[41, 45].

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. Uniform pressure distribution may be assumed in the calculation of the nominal bearing pressure between the base plate and the support. The maximum pressure on concrete foundations for factored loads should be limited to $0.6f_{cu}$, where f_{cu} is the 28-day cube strength of the concrete or the bedding material, whichever is the lesser. Holding down bolts and their anchorage should be generally designed to take tension as even if this does not occur due to the final loads it is likely to be necessary for stability under construction. The holes in the baseplate are usually oversized to allow for tolerance and washer plates are required.

15.1.2 Design of base plates

Base plates transmitting concentric loads may be designed by using an effective area, see Fig. 24.

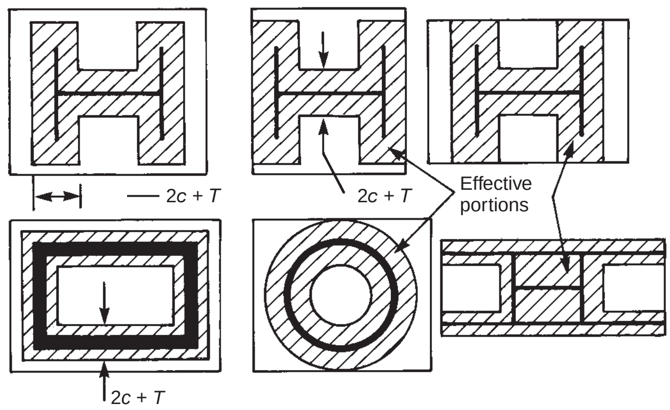


Fig. 24 Effective areas of base plates

For axial forces the thickness of the baseplate should not be less than t_p

where $t_p = c\left(\frac{3w}{p_{yb}}\right)^{0.5}$

t_p	is the thickness of the baseplate
c	is the projection of the effective portion of the baseplate beyond the face of the section
p_{yb}	is the design strength of the baseplate
w	is the pressure under the baseplate, based on an assumed uniform distribution of pressure throughout the effective portion.
T	is the flange thickness (or maximum thickness) of the column

Any overlap between the effective area between the flanges should be discounted in calculating the total effective area. This condition occurs when $c > D/2 - T$.

15.1.3 Design of gusset

The bending moments in any stiffening gusset should be limited to $p_{yg}\, Z_g$

where p_{yg} is the design strength of the gusset
Z_g is the appropriate elastic modulus of the gusset

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

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

15.2.1 Web cleats

a) Choose cleat size and calculate number and type of bolts. Typically, 90 x 90 x 10 thick angle, of length greater than 0.6 times the depth of the supported beam with M20 grade 8.8 bolts.

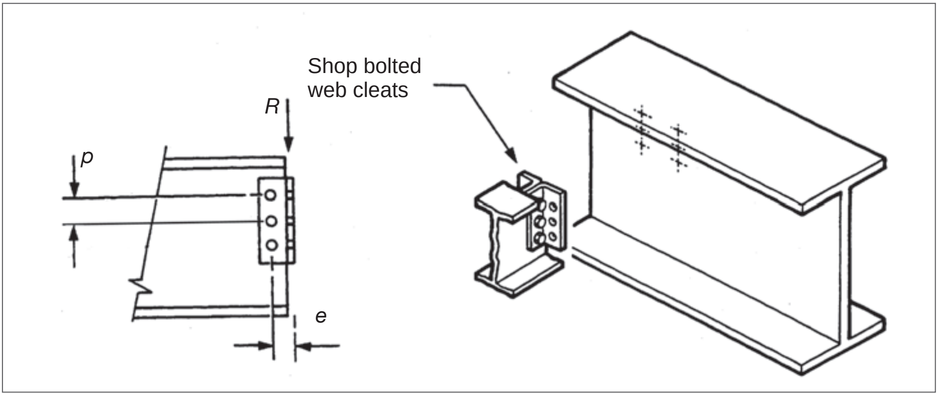


Fig. 25 Web cleat

b) Calculate force in the outermost bolts connecting the cleats to the beam web, from shear and eccentricity (see Note 2).
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 cleats to supporting column or beam.
f) Check bolts in single shear on supporting column or beam.
g) Check bearing stress on cleats and supporting column or beam taking account of beams framing in from both sides.
h) Check shear stress in cleats, supported beam and supporting column or beam (taking account of beams framing in from both sides) based on both gross and net area.
i) If there is a single or double notch, check reduced beam section for shear and moment, and local stability of notch. If the beam is unrestrained, check the overall stability of the notched beam.
j) Check block shear over appropriate cross-sections in cleats and supporting beams with notches, see Section 15.2.4.

Notes

1. Where maximum edge distances cannot be achieved the bolt strengths should be reduced proportionally.
2. Shear on bolt $= \sqrt{F_s^2 + F_b^2}$

where	F_s	=	reaction R/N and $F_b = Re/m$
	N	=	number of bolts
	m	=	modulus of bolt group = $N(N+1)p/6$
	p	=	bolt pitch
	R, e and p		are shown on Fig. 25

Where the beam is required to act as a tie for robustness, see Section 12, the connections must be able to transmit the appropriate tie force. This requires special consideration in the connection design. A number of 'structural integrity' checks are given in reference 45.

15.2.2 Thin flexible end plates welded only to web (see Fig. 26)

a) Choose plate size and calculate number and type of bolts. Typically, 150x8 thick plate for beams up to 457 and 200x10 thick for 533 beams and above with bolt cross-centres of 90 and 140 respectively. The length of the plate should normally be greater than 0.6 times the depth of the supported beam. Bolts are typically M20 grade 8.8.
b) Calculate force in bolts.
c) Check bolt strength in single shear.
d) Check bearing stress on plate and supporting column or beam taking account of beams framing in from both sides.
e) Check shear stress in plate and supporting column or beam (taking account of beams framing in from both sides) based on both gross and net area.
f) Check shear in the web of the supported beam over the depth of the end plate.

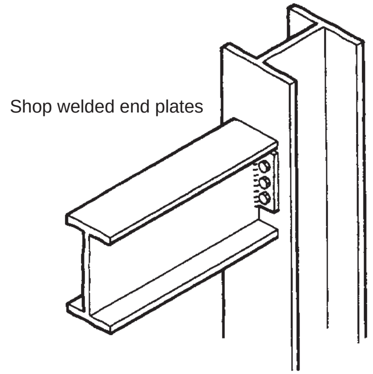


Fig. 26 Thin flexible end plates welded to web only

g) Choose fillet weld size to suit double length of weld (deducting amount equal to fillet weld leg length at each end of the run).

h) If there is a single or double notch, check reduced beam section for shear and moment, and local stability of notch. If the beam is unrestrained, check the overall stability of the notched beam.

i) Check block shear over appropriate cross-sections in plates (see Section 15.2.4).

Notes

1. Where maximum edge distances cannot be achieved the bolt strengths should be reduced proportionally.
2. Where the beam is required to act as a tie for robustness, see Section 12, the connections must be able to transmit the appropriate tie force. This requires special consideration in the connection design. A number of 'structural integrity' checks are given in reference 45.

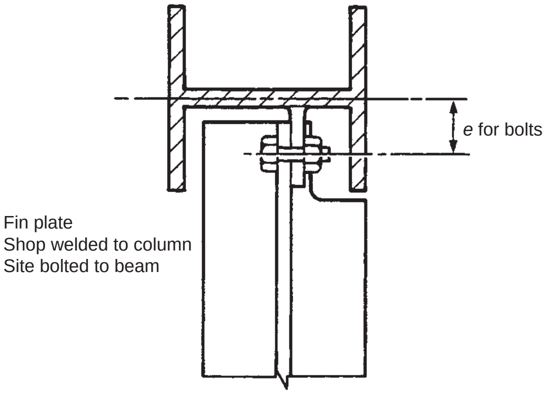


Fig. 27 Fin plate

15.2.3 Fin plates (see Fig. 27)

a) Choose fin plate size and number of bolts (grade 8.8 only). The length of the plate should normally be greater than 0.6 times the depth of the supported beam. Other restrictions also apply to ensure that the connection has adequate deformation capacity, (see Notes 3-5 below).

b) Calculate force in the outermost bolts connecting the fin plate to the beam web, from shear and eccentricity, (see Note 2 of 15.2.1).

c) Providing the limitations in Notes 4 and 5 are complied with, shear in the bolt will not govern the capacity.

d) Check bearing stress in web and fin plate.

e) Check shear stress in plate and supported beam based on both gross and net area.

f) Check shear stress in supporting column or beam (taking account of beams framing in from both sides). Confirm that punching shear is not limiting by ensuring that,

$$t_f \leqslant t_w \left(\frac{U_{sc}}{p_{yf}}\right)$$

where t_f is the fin plate thickness
t_w is the thickness of supporting web
U_{sc} is the ultimate tensile strength of supporting member
p_{yf} is the design strength of fin plate

g) Check shear and bending interaction of fin plate.

h) Ensure that the leg length of the weld is greater than or equal to 0.8 times the fin plate thickness.

i) If there is a single or double notch, check reduced beam section for shear and moment, and local stability of notch. If the beam is unrestrained, check the overall stability of the notched beam.

j) Check block shear over appropriate cross-sections in plate and supporting beams with notches, (see Section 15.2.4).

Notes

1. One vertical line of bolts only
2. Short fin plates only, eccentricity $\leqslant t_f/0.15$
3. Limit of beam depth to be 610 UB
4. t_p or t_w $\leqslant 0.50d_b$ for grade S275
   $\leqslant 0{\cdot}42d_b$, for grade S355
   where d_b = bolt diameter
5. Edge and end distance to be $\geqslant 2d_b$

15.2.4 Block shear failure

The possibility of failure in a connection caused by a block of material within the bolted area breaking away from the remainder of the section should be checked in certain cases. This is known as block shear failure and some typical cases are shown in Fig. 28.

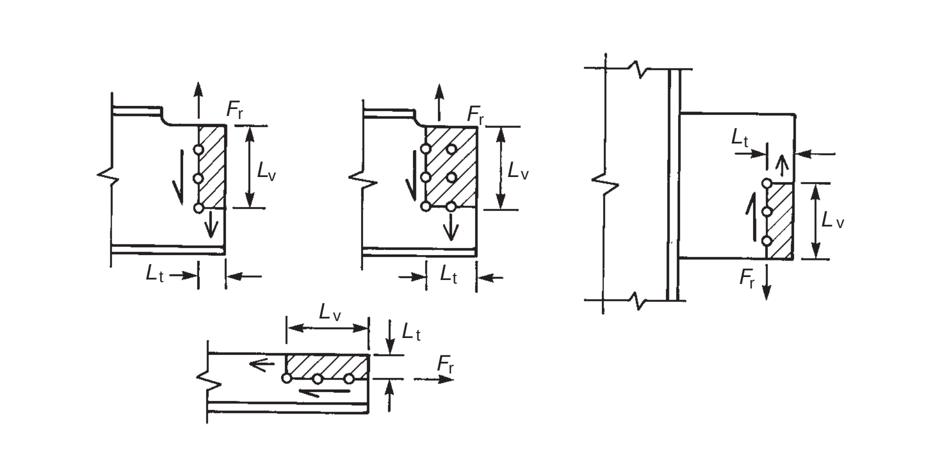


Fig. 28 Typical cases of block shear failure

This type of failure should be prevented by ensuring that the reaction F_r does not exceed the block shear capacity given by the following equation:

$$P_r = 0.6 p_y t [L_v + K_e (L_t - kD_t)]$$

where P_r is the block shear capacity
p_y is the design strength of the web or plate
t is the web or plate thickness
L_v is the length of the shear face
K_e is the effective net area coefficient, see Section 11.4.1
L_t is the length of the tension face
k is a coefficient, 0.5 for a single line of bolts and 2.5 for a double line
D_t is the is the hole size for the tension face, generally the hole diameter, but for slotted holes the dimension perpendicular to the direction of load transfer should be used

15.3 Column-to-column splices

Column splices are usually located adjacent to the floor level (typically 500mm above). At this position any moments due to strut action are considered insignificant. Splices should meet the following requirements:

- they should hold the connected members in place
- the centroidal axis of the splice should coincide with the centroidal axis of the connected members
- they should provide continuity of stiffness about both axes and should also resist any tension
- they should provide the tensile forces to comply with the robustness requirements of Section 12.

15.3.1 Column splices (ends prepared for contact in bearing) (see Fig. 29)

Splices detailed for full contact in bearing (see Section 14.1b)) should meet the following requirements:

- the projection of the flange cover plates beyond the ends of the column member should be equal to the width of the flange of the upper column or 225mm, whichever is greater
- 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
- when the column sections to be joined are of the same serial size, then nominal web cover plates may be used. These should incorporate at least four M20 grade 8.8 bolts and be of width ⩾ half the depth of the upper column
- when the column sections to be joined are of different serial size, then web cleats and a division plate should be used to give a load dispersion of 45º, see Fig. 29. The width of the web cleats should be ⩾ half the depth of the upper column.

It should be noted that the value of the moment and/or the sign of the axial force could create a net tension force in the cover plates. Tension may also exist due to the requirements for disproportionate collapse, see Section 12. In either case the tensile capacity of the cover plate based on gross and net area must be checked and the bolt group checked for shear and bearing. In the former check the effects of long joints and long grip lengths may need to be considered, see Table 17. Except when considering tension due to tying requirements, if the tensile force in the cover plate is greater than 10% of the capacity of the upper column flange then preloaded bolts designed to be non-slip under factored loads should be used.

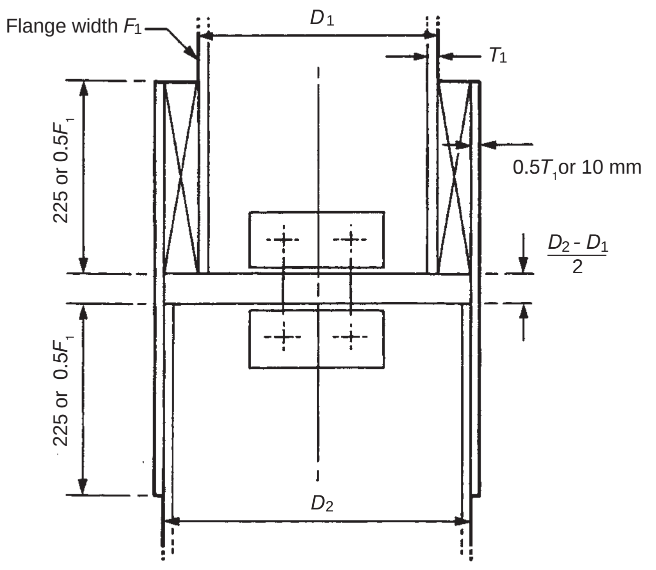


Fig. 29 Column splice – ends prepared for contact in bearing

15.3.2 Column-to-column splices (ends not prepared for contact in bearing) (see Fig. 30)

All the forces and moments should be wholly transmitted through the bolts, splice plates and web plates, and not by bearing between the members. A gap may be detailed between the two column sections to make it visually clear that the particular splice is non-bearing.

The design procedure is as follows.

a) Choose the splice plate size, number and type of bolts. A useful start point is to follow the recommendations for bearing splices.

b) Calculate the forces in the flange cover plates due to bending and axial force and due to tying requirements if appropriate.

c) For the flange cover plates, check the tension capacity based on gross and net area and the compression capacity based on gross area.

d) Check the bolt group in shear taking account of any reduction for long joints and long grip length, and in bearing, (see Table 17).

e) Check the web cover plates and their associated bolts in shear and bearing for the proportion of the axial load that is assigned to the web.

Notes

1. The axial force is carried by the web and the flanges.
2. The bending moment is deemed to be carried by the flanges.
3. The bolt diameter should be at least 75% of the packing thickness and the number of plies in multiple packs should not exceed four.
4. There should be not more than one change in serial size between lower and upper column.
5. Preloaded bolts should be provided if slip of the connections, which could lead to increased or excessive deflection, is not acceptable.

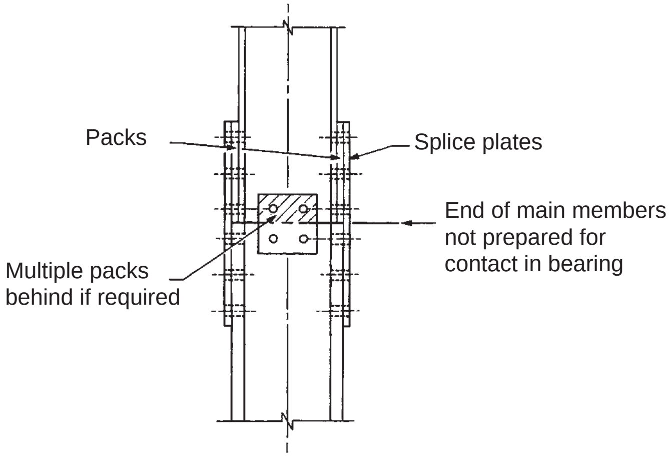


Fig. 30 Column splice – ends not prepared for contact in bearing

15.4 Portal frame connections

15.4.1 Portal frame – haunch

a) Assume the number and type of bolts required at 1 and 2 (see Fig. 31) to resist the factored bending moment, and locate them to obtain the maximum lever arm.

b) Using the force distribution shown in Fig. 22, 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. Prying forces may be ignored providing the 'simple method' for bolt tension capacity is used, see Section 14.2.2.

d) Check that the sum of the horizontal bolt forces, including any added at step b), can be resisted by the column in shear.

e) Check the bending stresses in the column flange caused by the bolt tension, assuming double curvature bending as in c). A local stiffener may need to be added.

f) Calculate the additional bolts required at 3 (see Fig. 31) 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 column flange.

g) Check for reversal of moment due to wind, as this may govern the design of bolts and of the stiffeners.

h) Check the haunch flange in bearing, where it meets the end plate. The flange bearing stress may be taken as $1.4p_y$. This is justified by two localised effects, strain hardening and some dispersion into the web at the root of the section.

i) Opposite the haunch bottom flange, check the column web and, if present, the stiffener in bearing and buckling.

j) Where the flange of the haunch meets the rafter, check the rafter web in bearing and buckling.

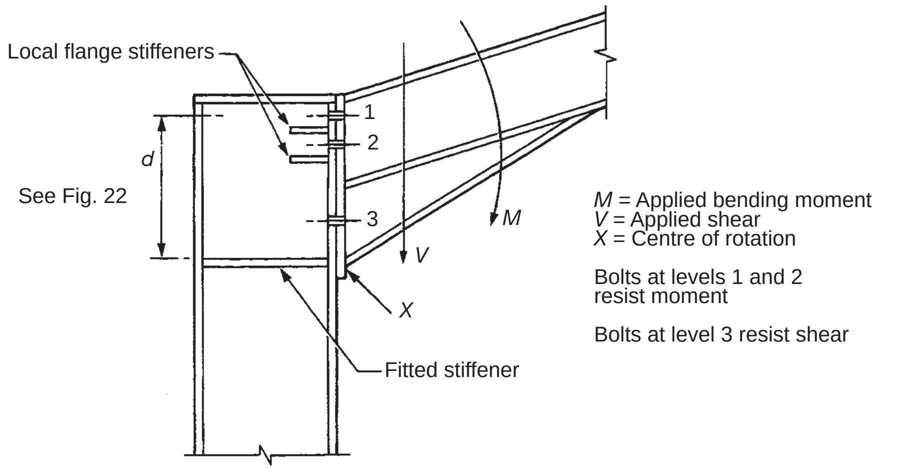


Fig. 31 Portal frame – haunch

15.4.2 Portal frame – ridge

a) Assume the number and type of bolts required at 1 and 2 (see Fig. 32) to resist the bending moment and locate them to obtain maximum lever arm.

b) Using the distribution of force shown in Fig. 22, calculate the resisting moment. If it is less than the applied moment increase the number 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. Prying forces may be ignored providing the 'simple method' for bolt tension capacity is used, see Section 14.2.2.

d) Calculate the bolts required to resist the applied vertical shear at 3 (see Fig. 32) by checking their single shear capacity and the bearing capacities on the end plates.

e) Check for reversal of moment due to wind.

f) Check the beam flange in bearing, where it meets the end plate. The flange bearing stress may be taken as $1.4p_y$. This is justified by two localised effects; strain hardening and some dispersion into the web at the root of the section.

g) Where the flange of the haunch meets the rafter, check the rafter web in bearing and buckling.

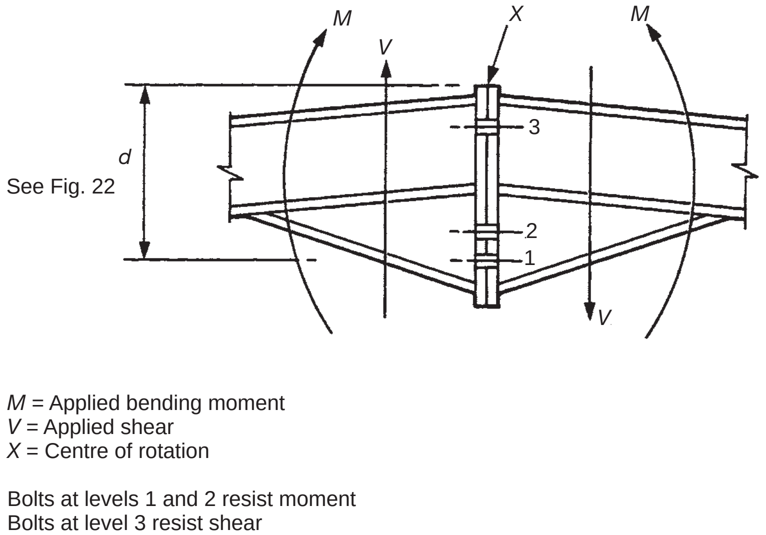


Fig. 32 Portal frame – ridge

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 force from a beam or haunch in a moment connection.

The bearing capacity and buckling resistance of an unstiffened web should be checked using the formulae given below. Where stiffeners are required, reference should be made to clauses 4.5.2 and 4.5.3 of BS 5950-1:2000[1].

$$P_{bw} = (b_1 + nk)t_w p_{yw}$$

where P_{bw} is the bearing capacity of the web

b_1 is the stiff bearing length taken as the length of support or components applying the load that cannot deform appreciably in bending. Some examples are shown in Fig. 33

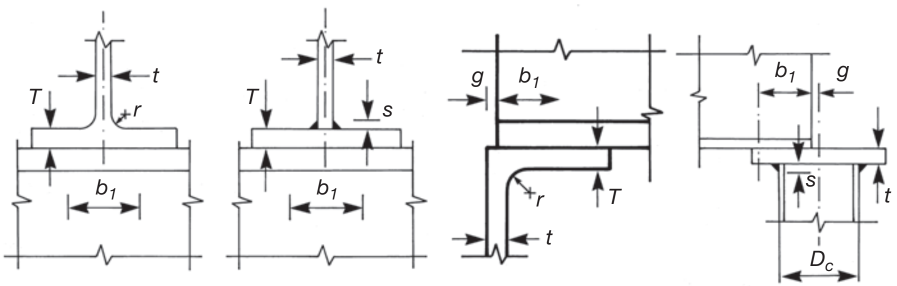
n = 5, except at the end of a member where $n = 2 + 0.6\, b_e/k$ but $\leqslant 5$

k = $T + r$ for a rolled I or H section or $k = T$ for a welded I- or H- section

t_w is the thickness of the web of the member being checked for bearing

p_{yw} is the design strength of the web

b_e is the distance to the nearer end of the member from the end of the stiff bearing



$b_1 = t + 1.6r + 2T$ $\quad$ $b_1 = t + 1.6s + 2T$ $\quad$ $b_1 = t + T + 0.8r - g$ $\quad$ $b_1 = 0.5D_c + t + 0.8_s - g$

Fig. 33 Stiff bearing length

The buckling capacity of the web P_x should be calculated from the following formula provided that;

a) The distance from the centre of the load or reaction to the end of the web is not less than 0.7*d*.

b) The flange through which the load or reaction is applied is effectively restrained against both rotation relative to the web and lateral movement relative to the flange.

$$P_x = \frac{25\varepsilon t_w}{\sqrt{(b_1 + nk)d}} P_{bw}$$

Where a) is not satisfied then the buckling capacity is reduced to:

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\varepsilon t_w}{\sqrt{(b1 + nk)d}} P_{bw}$$

where d is the depth of the web

a_e is the distance from the centre of the load or reaction to the nearer end of the web

ε $= \sqrt{\frac{275}{p_y}}$

Where b) is not satisfied then the buckling capacity is reduced to P_{xr}, given by:

$$P_{xt} = \frac{0.7d\ P_x}{L_E}$$

where P_X is the value obtained from the formulae above depending upon whether the condition a) above is met

L_E is the effective length of the web acting as a compression member determined in accordance with Table 22 of BS 5950-1:2000[1].

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Appendix A
Moment capacities M_{cx} for fully restrained beam, 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 S275 steel

Serial size	Mass (kg/m)	I x-x axis (cm⁴)	Max M_{cx} fully restrained (kNm)	Critical values of L_E (m)	Buckling resistance moments for effective lengths between restraints of L_E (m)						
					4	5	6	7	8	9	10
1016 x 305	487	1020 000	5920	2.77	5280	4770	4300	3890	3530	3220	2960
	437	910 000	5290	2.71	4670	4190	3750	3360	3030	2750	2510
	393	808 000	4730	2.66	4120	3670	3260	2900	2590	2330	2120
	349	723 000	4400	2.65	3780	3350	2940	2590	2300	2050	1850
	314	644 000	3940	2.55	3350	2950	2570	2250	1980	1750	1570
	272	554 000	3400	2.54	2870	2510	2170	1880	1640	1440	1280
	249	481 000	3010	2.45	2490	2160	1850	1580	1370	1190	1050
	222	408 000	2600	2.37	2100	1800	1520	1290	1100	954	837
914 x 419	388	720 000	4680	3.81	4650	4320	3990	3680	3380	3100	2850
	343	626 000	4100	3.75	4050	3750	3460	3170	2890	2640	2400
914 x 305	289	504 000	3300	2.62	2870	2530	2220	1940	1700	1510	1350
	253	436 000	2900	2.59	2470	2160	1880	1630	1420	1250	1110
	224	376 000	2530	2.53	2130	1850	1590	1360	1180	1030	905
	201	325 000	2210	2.46	1830	1580	1340	1140	977	847	742
838 x 292	226	340 000	2430	2.51	2040	1780	1540	1330	1160	1020	907
	194	279 000	2020	2.44	1670	1440	1230	1050	904	787	693
	176	246 000	1800	2.39	1470	1250	1060	899	769	665	583
762 x 267	197	240 000	1900	2.30	1520	1310	1120	961	836	736	656
	173	205 000	1640	2.24	1290	1100	928	790	680	594	525
	147	169 000	1370	2.18	1050	880	733	616	524	453	397
	134	151 000	1280	2.15	959	794	655	545	460	395	344
686 x 254	170	170 000	1490	2.22	1170	1000	856	736	641	565	505
	152	150 000	1330	2.18	1030	870	735	627	541	474	421
	140	136 000	1210	2.15	926	779	654	553	475	414	366
	125	118 000	1060	2.11	795	661	549	460	392	339	298
610 x 305	238	210 000	1980	2.92	1810	1640	1480	1340	1220	1110	1020
	179	153 000	1470	2.81	1310	1170	1040	924	823	737	665
	149	126 000	1220	2.76	1070	953	838	736	648	574	513
610 x 229	140	112 000	1100	2.01	813	684	578	495	431	381	341
	125	98 600	974	1.98	709	591	494	420	362	318	283
	113	87 300	869	1.95	621	512	424	358	305	266	236
	101	75 800	792	1.91	545	442	361	300	254	220	193
533 x 210	122	76 000	847	1.87	602	504	427	368	322	286	257
	109	66 800	749	1.84	520	430	360	307	267	235	211
	101	61 500	692	1.83	474	389	324	274	237	208	186
	92	55 200	649	1.80	430	349	286	240	206	180	159
	82	47 500	566	1.76	364	291	236	196	167	144	127
457 x 191	98	45 700	591	1.73	401	334	283	245	215	192	173
	89	41 000	534	1.72	353	291	245	209	183	162	146
	82	37 100	504	1.68	322	261	217	184	159	141	126
	74	33 300	455	1.67	284	229	188	158	136	120	106
	67	29 400	405	1.64	246	196	159	133	113	99	87.7

Table A2 S275 steel												
Serial size	Mass (kg/m)	I x-x axis (cm⁴)	Max M_{cx} fully restrained (kNm)	Critical values of L_e (m)	Buckling resistance moments for effective lengths between restraints of L_e (m)							
					2	2.5	3	3.5	4	5	6	7
457 x 152	82	36 600	480	1.36	422	376	333	295	263	212	177	152
	74	32 700	431	1.34	375	333	293	257	227	182	150	128
	67	28 900	400	1.32	342	300	261	227	198	156	127	107
	60	25 500	354	1.29	300	262	226	195	169	131	106	88.5
	52	21 400	301	1.25	250	216	184	157	135	103	81.9	67.8
406 x 178	74	27 300	413	1.61	386	351	318	286	257	210	176	150
	67	24 300	370	1.59	344	312	281	251	224	181	150	127
	60	21 600	330	1.57	305	276	247	220	195	156	128	107
	54	18 700	290	1.53	266	239	212	188	165	130	105	87.7
406 x 140	46	15 700	244	1.21	199	171	146	124	107	82.3	66.3	55.3
	39	12 500	199	1.16	157	133	111	93.7	79.6	60	47.5	39.1
356 x 171	67	19 500	333	1.59	310	283	257	232	210	174	147	127
	57	16 000	278	1.56	256	232	208	187	167	135	112	95.7
	51	14 100	246	1.53	226	203	182	161	143	115	94.1	79.5
	45	12 100	213	1.5	193	173	153	135	119	93.5	75.9	63.4
356 x 127	39	10 200	181	1.07	138	116	97.2	82.5	71.1	55.2	45	38
	33	8 250	149	1.04	110	90.9	75.2	62.9	53.5	40.7	32.7	27.3
305 x 165	54	11 700	233	1.57	216	196	178	161	146	121	102	88.6
	46	9 900	198	1.54	182	165	148	133	119	96.8	80.8	69.2
	40	8 500	171	1.52	156	141	126	112	99.3	79.5	65.5	55.4
305 x 127	48	9 580	196	1.11	156	136	118	104	92.4	75.3	63.5	55
	42	8 200	169	1.09	132	113	97.4	84.7	74.5	59.8	49.9	42.9
	37	7 170	148	1.07	114	96.6	82.2	70.7	61.7	48.8	40.4	34.5
305 x 102	33	6 500	132	0.87	86.4	70.5	58.5	49.6	43	33.9	28	23.9
	28	5 370	111	0.84	69.3	55.4	45.2	37.9	32.4	25.1	20.5	17.4
	25	4 460	94.1	0.81	55.7	43.8	35.2	29.1	24.7	18.9	15.3	12.9
254 x 146	43	6 540	156	1.41	139	125	113	102	92	76.5	65.2	56.8
	37	5 540	133	1.39	117	105	93.5	83.3	74.5	60.9	51.2	44.2
	31	4 410	108	1.34	93.5	82.7	72.6	63.7	56.1	44.8	37	31.4
254 x 102	28	4 010	97.1	0.89	65.9	54.7	46	39.5	34.5	27.8	23	19.8
	25	3 420	84.2	0.87	55	44.9	37.3	31.6	27.4	21.6	17.9	15.3
	22	2 840	71.2	0.84	44.5	35.6	29.2	24.5	21	16.3	13.4	11.4
203 x 133	30	2 900	86.4	1.29	74.1	66.1	58.9	52.6	47.3	39.1	33.2	28.9
	25	2 340	71	1.25	59.6	52.5	46	40.5	35.9	29	24.3	20.9

Table A3 S355 steel

Serial size	Mass (kg/m)	I x-x axis (cm⁴)	Max M_{cx} fully restrained (kNm)	Critical values of L_e (m)	Buckling resistance moments for effective lengths between restraints of L_e (m) 4	5	6	7	8	9	10
1016 x 305	487	1020 000	7770	2.4	6580	5850	5180	4590	4100	3690	3360
	437	910 000	6950	2.36	5810	5120	4490	3950	3500	3130	2830
	393	808 000	6210	2.32	5120	4470	3880	3380	2970	2640	2370
	349	723 000	5720	2.31	4660	4040	3470	2990	2610	2300	2050
	314	644 000	5120	2.24	4130	3540	3020	2580	2230	1950	1730
	272	554 000	4430	2.23	3530	3010	2540	2150	1840	1600	1410
	249	481 000	3920	2.15	3060	2570	2140	1800	1520	1310	1150
	222	408 000	3380	2.08	2570	2130	1750	1450	1220	1050	909
914 x 419	388	720 000	6100	3.34	5830	5380	4920	4460	4030	3640	3300
	343	626 000	5340	3.29	5080	4670	4250	3830	3440	3080	2770
914 x 305	289	504 000	4340	2.3	3540	3050	2610	2230	1930	1690	1490
	253	436 000	3770	2.27	3040	2600	2200	1860	1590	1380	1220
	224	376 000	3290	2.22	2610	2210	1850	1550	1320	1130	990
	201	325 000	2880	2.16	2240	1880	1550	1290	1090	930	808
838 x 292	226	340 000	3160	2.2	2510	2130	1800	1520	1300	1130	995
	194	279 000	2640	2.14	2050	1710	1420	1190	1010	866	756
	176	246 000	2350	2.1	1790	1490	1220	1010	853	729	633
762 x 267	197	240 000	2470	2.01	1850	1550	1290	1090	930	809	715
	173	205 000	2140	1.97	1570	1290	1060	887	753	650	570
	147	169 000	1780	1.91	1270	1030	835	687	576	493	429
	134	151 000	1650	1.89	1150	919	737	602	501	426	369
686 x 254	170	170 000	1940	1.94	1420	1180	984	829	711	621	550
	152	150 000	1730	1.92	1240	1020	841	702	598	518	456
	140	136 000	1570	1.89	1120	912	745	618	523	451	396
	125	118 000	1380	1.85	956	770	622	511	429	368	320
610 x 305	238	210 000	2580	2.54	2240	2010	1780	1590	1420	1270	1160
	179	153 000	1910	2.46	1620	1430	1240	1080	942	832	742
	149	126 000	1580	2.42	1330	1160	993	852	736	643	568
610 x 229	140	112 000	1430	1.76	975	796	656	552	475	415	369
	125	98 600	1270	1.74	849	685	559	466	397	345	305
	113	87 300	1130	1.71	740	590	477	394	334	288	253
	101	75 800	1020	1.68	642	503	401	328	275	236	206
533 x 210	122	76 000	1100	1.64	716	583	482	409	354	312	278
	109	66 800	976	1.61	616	494	404	339	291	255	227
	101	61 500	901	1.6	561	446	362	302	258	225	200
	92	55 200	838	1.58	503	395	317	262	222	193	170
	82	47 500	731	1.55	424	328	261	213	179	154	135
457 x 191	98	45 700	770	1.52	473	383	318	271	235	208	187
	89	41 000	695	1.5	415	333	273	231	199	176	157
	82	37 100	650	1.48	374	295	240	201	172	151	134
	74	33 300	587	1.47	329	257	207	172	147	128	113
	67	29 400	522	1.44	284	219	175	144	122	106	93.1

Table A4 S355 steel												
Serial size	Mass	I x-x axis	Max M_{cx} fully restrained	Critical values of L_e	Buckling resistance moments for effective lengths between restraints of L_e (m)							
	(kg/m)	(cm^4)	(kNm)	(m)	2	2.5	3	3.5	4	5	6	7
457 x 152	82	36 600	625	1.19	522	456	395	343	300	237	194	165
	74	32 700	561	1.17	464	402	346	297	258	201	164	138
	67	28 900	516	1.15	418	359	304	259	223	171	138	115
	60	25 500	457	1.13	366	312	262	221	189	143	114	94.6
	52	21 400	389	1.1	305	256	213	177	150	112	87.8	72.1
406 x 178	74	27 300	533	1.41	478	430	382	337	298	237	194	164
	67	24 300	478	1.39	426	381	336	295	259	203	165	138
	60	21 600	426	1.38	377	336	295	257	224	174	140	116
	54	18 700	375	1.34	328	291	253	218	189	144	115	94.5
406 x 140	46	15 700	315	1.06	242	202	167	140	118	89.3	71	58.8
	39	12 500	257	1.02	190	156	127	105	87.5	64.6	50.6	41.4
356 x 171	67	19 500	430	1.39	384	346	309	274	244	196	163	139
	57	16 000	359	1.36	317	283	249	219	192	151	124	104
	51	14 100	318	1.34	279	247	217	188	164	128	103	86
	45	12 100	275	1.31	238	210	182	157	135	104	82.7	68.3
356 x 127	39	10 200	234	0.94	165	134	110	91.8	78	59.5	48	40.2
	33	8 250	193	0.91	131	105	84.5	69.4	58.3	43.7	34.7	28.8
305 x 165	54	11 700	300	1.37	267	240	214	190	169	136	113	96.9
	46	9 900	256	1.35	225	201	177	156	137	108	88.9	75.2
	40	8 500	221	1.33	193	171	150	130	114	88.6	71.7	60
305 x 127	48	9 580	252	0.97	188	160	136	118	103	82.5	68.8	59.1
	42	8 200	218	0.95	159	133	112	95.1	82.6	65.1	53.8	45.9
	37	7 170	191	0.94	136	113	93.6	79	68	52.9	43.3	36.7
305 x 102	33	6 500	171	0.76	101	79.8	64.9	54.3	46.5	36.2	29.7	25.2
	28	5 370	143	0.74	80.3	62.3	49.8	41.1	34.9	26.7	21.7	18.3
	25	4 460	121	0.71	64	48.8	38.5	31.5	26.5	20	16.1	13.5
254 x 146	43	6 540	201	1.23	171	152	134	119	106	85.8	72	62
	37	5 540	171	1.21	144	127	111	96.5	84.9	67.8	56.1	47.9
	31	4 410	140	1.17	115	99.2	85.1	73.1	63.3	49.3	40.1	33.8
254 x 102	28	4 010	125	0.78	77.4	62.3	51.4	43.5	37.6	29.6	24.5	20.9
	25	3 420	109	0.76	64.2	50.8	41.3	34.6	29.6	23.1	18.9	16.1
	22	2 840	91.9	0.74	51.5	40	32.1	26.6	22.6	17.4	14.1	11.9
203 x 133	30	2 900	111	1.12	90.7	79.3	69.2	60.7	53.7	43.4	36.4	31.4
	25	2 340	91.6	1.09	72.8	62.6	53.6	46.3	40.4	32	26.4	22.5

Appendix B
Bending strength, p_b, for rolled sections

Table B1 – Steel Grade S275					
λ_{LT}	Design strength p_y (N/mm²)				
	235	**245**	**255**	**265**	**275**
25	235	245	255	265	275
30	235	245	255	265	275
35	235	245	255	265	273
40	229	238	246	254	262
45	219	227	235	242	250
50	210	217	224	231	238
55	199	206	213	219	226
60	189	195	201	207	213
65	179	185	190	196	201
70	169	174	179	184	188
75	159	164	168	172	176
80	150	154	158	161	165
85	140	144	147	151	154
90	132	135	138	141	144
95	124	126	129	131	134
100	116	118	121	123	125
105	109	111	113	115	117
110	102	104	106	107	109
115	96	97	99	101	102
120	90	91	93	94	96
125	85	86	87	89	90
130	80	81	82	83	84
135	75	76	77	78	79
140	71	72	73	74	75
145	67	68	69	70	71
150	64	64	65	66	67
155	60	61	62	62	63
160	57	58	59	59	60
165	54	55	56	56	57
170	52	52	53	53	54
175	49	50	50	51	51
180	47	47	48	48	49
185	45	45	46	46	46
190	43	43	44	44	44
195	41	41	42	42	42
200	39	39	40	40	40
210	36	36	37	37	37
220	33	33	34	34	34
230	31	31	31	31	31
240	28	29	29	29	29
250	26	27	27	27	27
λ_{L0}	**37.1**	**36.3**	**35.6**	**35**	**34.3**

Table B2 – Steel Grade S355

λ_{LT}	Design strength p_y (N/mm²)				
	315	**325**	**335**	**345**	**355**
25	315	325	335	345	355
30	315	325	335	345	355
35	307	316	324	332	341
40	294	302	309	317	325
45	280	287	294	302	309
50	265	272	279	285	292
55	251	257	263	268	274
60	236	241	246	251	257
65	221	225	230	234	239
70	206	210	214	218	222
75	192	195	199	202	205
80	178	181	184	187	190
85	165	168	170	173	175
90	153	156	158	160	162
95	143	144	146	148	150
100	132	134	136	137	139
105	123	125	126	128	129
110	115	116	117	119	120
115	107	108	109	110	111
120	100	101	102	103	104
125	94	95	96	96	97
130	88	89	90	90	91
135	83	83	84	85	85
140	78	78	79	80	80
145	73	74	74	75	75
150	69	70	70	71	71
155	65	66	66	67	67
160	62	62	63	63	63
165	59	59	59	60	60
170	56	56	56	57	57
175	53	53	53	54	54
180	50	51	51	51	51
185	48	48	48	49	49
190	46	46	46	46	47
195	43	44	44	44	44
200	42	42	42	42	42
210	38	38	38	39	39
220	35	35	35	35	36
230	32	32	33	33	33
240	30	30	30	30	30
250	28	28	28	28	28
λ_{L0}	**32.1**	**31.6**	**31.1**	**30.6**	**30.2**

Table B3 – Steel Grade S460					
λ_{LT}	Design strength p_y (N/mm²)				
	400	**410**	**430**	**440**	**460**
25	400	410	430	440	460
30	395	403	421	429	446
35	378	386	402	410	426
40	359	367	382	389	404
45	340	347	361	367	381
50	320	326	338	344	356
55	299	305	315	320	330
60	278	283	292	296	304
65	257	261	269	272	279
70	237	241	247	250	256
75	219	221	226	229	234
80	201	203	208	210	214
85	185	187	190	192	195
90	170	172	175	176	179
95	157	158	161	162	164
100	145	146	148	149	151
105	134	135	137	138	140
110	124	125	127	128	129
115	115	116	118	118	120
120	107	108	109	110	111
125	100	101	102	103	104
130	94	94	95	96	97
135	88	88	89	90	90
140	82	83	84	84	85
145	77	78	79	79	80
150	73	73	74	74	75
155	69	69	70	70	71
160	65	65	66	66	67
165	61	62	62	62	63
170	58	58	59	59	60
175	55	55	56	56	56
180	52	53	53	53	54
185	50	50	50	51	51
190	48	48	48	48	48
195	45	45	46	46	46
200	43	43	44	44	44
210	39	40	40	40	40
220	36	36	37	37	37
230	33	33	34	34	34
240	31	31	31	31	31
250	29	29	29	29	29
λ_{L0}	**28.4**	**28.1**	**27.4**	**27.1**	**26.5**

Appendix C
Compression resistance, buckling resistance moment and minor axis capacity of UC columns (S355 steel)

UC section – S355		Storey height (m)				
		0	3	4	5	6
356 x 406 x 634	$A_g p_c$	26300	24700	23000	21400	19700
$p_y Z_y$ = 1500	M_{bs}	4520	4520	4520	4520	4520
356 x 406 x 551	$A_g p_c$	22800	21400	19900	18500	17000
$p_y Z_y$ = 1280	M_{bs}	3890	3890	3890	3890	3890
356 x 406 x 467	$A_g p_c$	19900	18600	17300	16000	14700
$p_y Z_y$ = 1100	M_{bs}	3350	3350	3350	3350	3350
356 x 406 x 393	$A_g p_c$	16800	15600	14500	13400	12300
$p_y Z_y$ = 912	M_{bs}	2750	2750	2750	2750	2750
356 x 406 x 340	$A_g p_c$	14500	13400	12500	11500	10500
$p_y Z_y$ = 779	M_{bs}	2340	2340	2340	2340	2340
356 x 406 x 287	$A_g p_c$	12600	11900	11300	10600	9850
$p_y Z_y$ = 669	M_{bs}	2010	2010	2010	2010	2010
356 x 406 x 235	$A_g p_c$	10300	9750	9220	8650	8030
$p_y Z_y$ = 542	M_{bs}	1620	1620	1620	1620	1620
356 x 368 x 202	$A_g p_c$	8870	8300	7810	7280	6690
$p_y Z_y$ = 436	M_{bs}	1370	1370	1370	1370	1360
356 x 368 x 177	$A_g p_c$	7800	7270	6840	6370	5850
$p_y Z_y$ = 380	M_{bs}	1190	1190	1190	1190	1180
356 x 368 x 153	$A_g p_c$	6730	6270	5900	5490	5040
$p_y Z_y$ = 327	M_{bs}	926	926	926	926	918
356 x 368 x 129	$A_g p_c$	5660	5280	4970	4620	4230
$p_y Z_y$ = 274	M_{bs}	781	781	781	781	773
305 x 305 x 283	$A_g p_c$	12100	10600	9560	8530	7510
$p_y Z_y$ = 512	M_{bs}	1710	1710	1710	1710	1640
305 x 305 x 240	$A_g p_c$	10600	9560	8840	8030	7150
$p_y Z_y$ = 440	M_{bs}	1470	1470	1470	1460	1390
305 x 305 x 198	$A_g p_c$	8690	7870	7260	6580	5840
$p_y Z_y$ = 358	M_{bs}	1190	1190	1190	1180	1120
305 x 305 x 158	$A_g p_c$	6930	6250	5760	5200	4600
$p_y Z_y$ = 279	M_{bs}	925	925	925	917	867
305 x 305 x 137	$A_g p_c$	6000	5410	4970	4480	3950
$p_y Z_y$ = 239	M_{bs}	792	792	792	784	740

UC section – S355		Storey height (m)				
		0	3	4	5	6
305 x 305 x 118	$A_g p_c$	5180	4650	4270	3840	3380
$p_y Z_y$ = 203	M_{bs}	676	676	676	667	629
305 x 305 x 97	$A_g p_c$	4370	3910	3590	3210	2810
$p_y Z_y$ = 170	M_{bs}	513	513	513	503	474
254 x 254 x 167	$A_g p_c$	7350	6370	5720	4990	4240
$p_y Z_y$ = 257	M_{bs}	836	836	836	793	738
254 x 254 x 132	$A_g p_c$	5800	5000	4480	3890	3290
$p_y Z_y$ = 199	M_{bs}	645	645	645	608	565
254 x 254 x 107	$A_g p_c$	4690	4040	3600	3120	2630
$p_y Z_y$ = 158	M_{bs}	512	512	512	480	445
254 x 254 x 89	$A_g p_c$	3900	3350	2980	2580	2160
$p_y Z_y$ = 131	M_{bs}	422	422	422	395	366
254 x 254 x 73	$A_g p_c$	3310	2820	2500	2140	1790
$p_y Z_y$ = 109	M_{bs}	319	319	317	296	273
203 x 203 x 86	$A_g p_c$	3800	3010	2530	2050	1620
$p_y Z_y$ = 103	M_{bs}	337	337	317	289	258
203 x 203 x 71	$A_g p_c$	3120	2470	2070	1670	1320
$p_y Z_y$ = 84.9	M_{bs}	276	276	259	235	210
203 x 203 x 60	$A_g p_c$	2710	2120	1760	1400	1100
$p_y Z_y$ = 71.4	M_{bs}	233	233	216	196	174
203 x 203 x 52	$A_g p_c$	2350	1840	1520	1210	950
$p_y Z_y$ = 61.8	M_{bs}	201	201	187	169	150
203 x 203 x 46	$A_g p_c$	2080	1620	1340	1060	830
$p_y Z_y$ = 54.0	M_{bs}	160	160	148	133	118
152 x 152 x 37	$A_g p_c$	1670	1080	792	575	428
$p_y Z_y$ = 32.5	M_{bs}	110	102	88.4	74.2	60.9
152 x 152 x 30	$A_g p_c$	1360	868	633	459	341
$p_y Z_y$ = 26.0	M_{bs}	88	81.3	70.4	59	48.3
152 x 152 x 23	$A_g p_c$	1040	643	462	332	246
$p_y Z_y$ = 18.7	M_{bs}	58.2	53	45.5	37.7	30.6

Notes

a The buckling resistance $A_g p_c$ is the minimum of that about the x-x axis and that about the y-y axis based on effective length factors of 1.0 and 0.85 respectively

b The lateral torsional buckling resistance moment M_{bs} is based on an equivalent slenderness λ_{LT} of 0.5 L/r_y and is applicable to columns in simple construction only.

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-section with plates type B	Minor

Table D2 Compressive strength p_c for	
Section	**Axis of buckling**
Rolled I-sections	Minor
Rolled H-sections < 40mm flange	Major
Rolled, flat or square bars < 40mm	Major
Rolled I- or H-sections with plates type B	
up to 40mm	Major
over 40mm	Major

Table D3 Compressive strength p_c for	
Section	**Axis of buckling**
Rolled H-sections < 40mm	Minor
Rolled H-section > 40mm	Major
Rolled I- or H-sections with plates type B > 40mm	Major
Round, square or flat bars > 40mm thick	Major and minor
Rolled angles, channels or T-sections, compound,	
Laced or battened members	Any axis
Cold formed structural hollow sections	Any axis

Table D4 Compressive strength p_c for	
Section	**Axis of buckling**
Rolled H-sections > 40mm	Minor
Welded sections	Minor

Table D1 Compressive strength p_c for	
section	axis of buckling
hot-rolled hollow sections rolled I-sections rolled I-sections with plates type A rolled I- or H-section with plates type B	major and minor major major minor

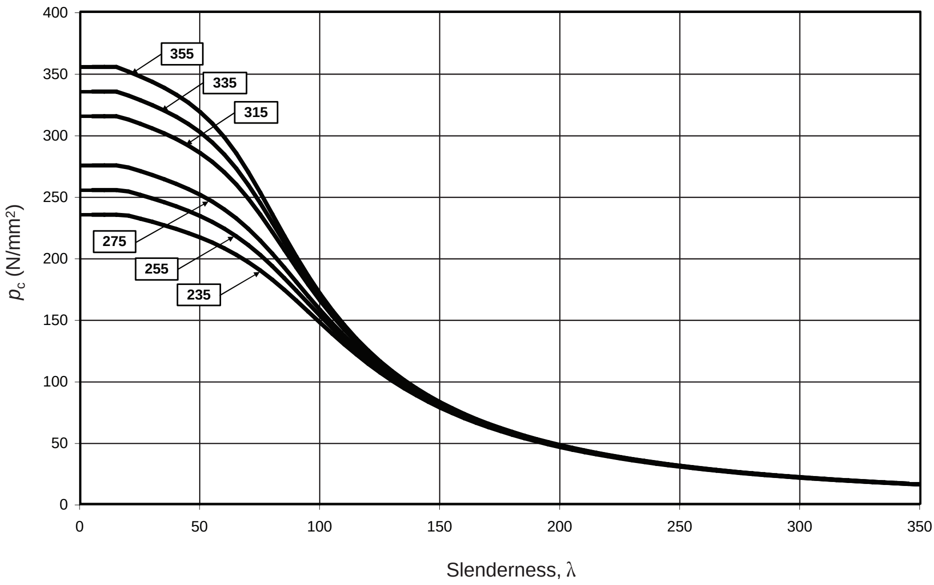


Table D2 Compressive strength p_c for	
section	axis of buckling
rolled I-sections rolled H-sections < 40mm flange rolled, flat or square bars < 40mm rolled I- or H-sections with plates type B up to 40mm over 40mm	minor major major major major

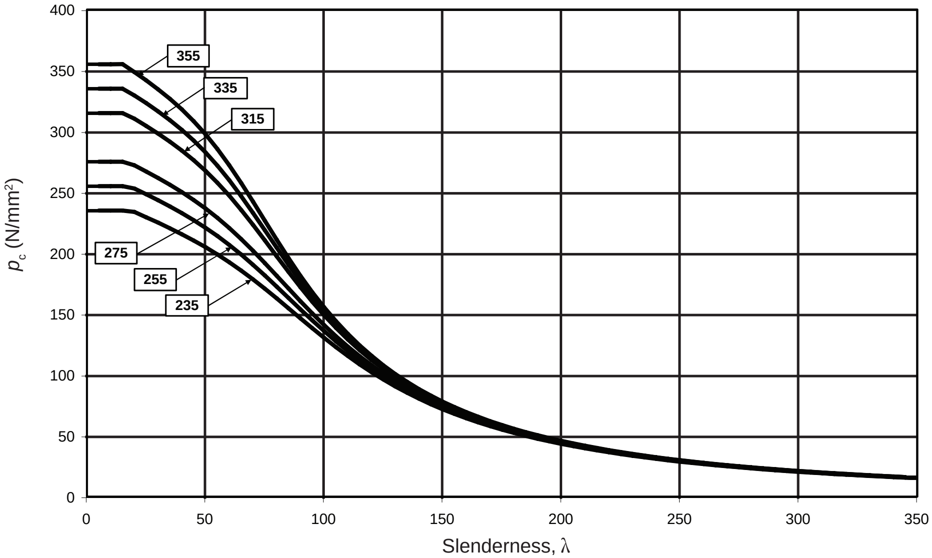


Table D3 Compressive strength p_c for	
section	axis of buckling
rolled H-sections < 40mm rolled H-section > 40mm rolled I- or H-sections with plates type B > 40mm round, square or flat bars > 40mm thick rolled angles, channels or T-sections, compound, laced or battened members cold formed structural hollow sections	minor major major major and minor any axis any axis

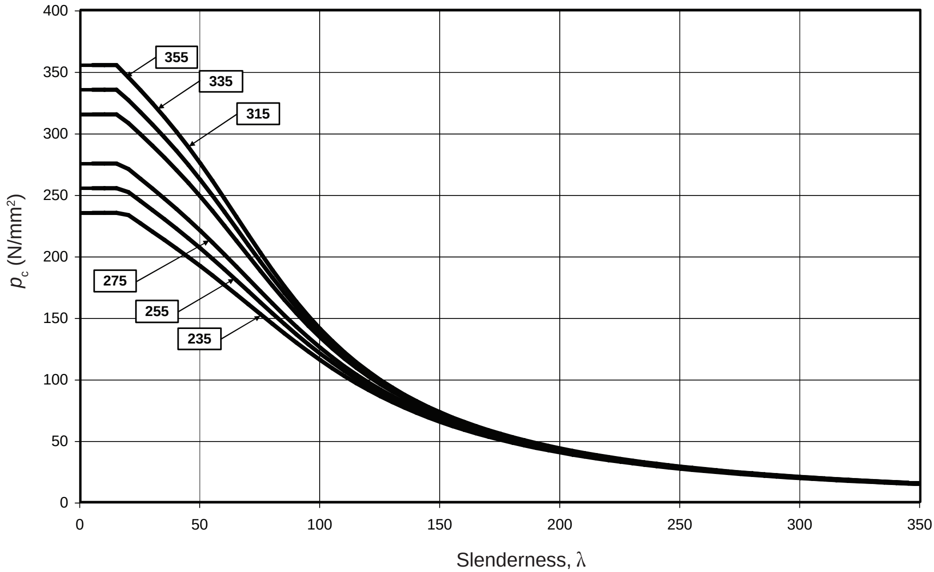
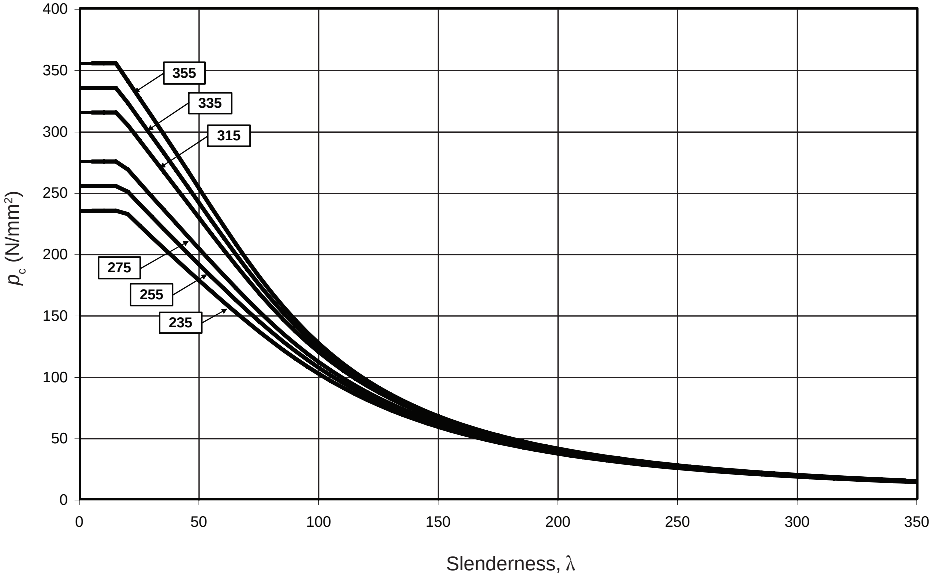


Table D4 Compressive strength p_c for	
section	axis of buckling
rolled H-sections > 40mm welded sections	minor minor



Appendix E
Design data

Contract **Job no.**

General description, intended use, unusual environmental conditions

Site constraints

Stability provisions

Movement joints

Loading

Fire resistance

Durability

Soil conditions and foundation design

Performance criteria

Materials

Ground slab construction

Other data