BRITISH STANDARD

BS 449-2: 1969

Incorporating amendments issued January 1970 (AMD 416), May 1970 (AMD 523), December 1970 (AMD 661), April 1973 (AMD 1135), July 1975 (AMD 1787), August 1984 (AMD 4576), May 1988 (AMD 5698), December 1989 (AMD 6255) and Amendment No. 9

Specification for

The use of structural steel in building —

Part 2: Metric units

UDC 693.814:624.016.7:669.14.018.29

Co-operating organizations

The Technical Committee of the Building Divisional Council responsible for the revision of this British Standard (now incorporating the British Standard Code of Practice CP 113:1948, "The structural use of steel in buildings"), consists of representatives from the following Government departments and scientific and industrial organizations, and of additional members nominated to represent the Institution of Structural Engineers Committee, under whose supervision CP 113:1948 was prepared:

Association of Municipal Corporations
British Constructional Steelwork Association
British Railways Board
British Steel Industry
Crown Agents for Oversea Governments and Administrations
District Surveyor's Association
Greater London Council
Imperial Chemical Industries Ltd.
Institute of Building
Institution of Civil Engineers
Institution of Municipal Engineers
Institution of Structural Engineers
Ministry of Public Building and Works
Ministry of Public Building and Works, Building Research Station
National Federation of Building Trades Employers
Royal Institute of British Architects
The Welding Institute

This British Standard, having been approved by the Building Divisional Council and the Council for Codes of Practice, was published under the authority of the Executive Board on 31st October, 1969

First published, April, 1932
First revision, December, 1935
Second revision, July, 1937
Third revision, July, 1948
Fourth revision (incorporating CP 113), May, 1959
Reset and reprinted, February, 1965
Reprinted in October, 1967
BS 449-2, metric edition of revision, published October, 1969

The following BSI references relate to the work on this standard:
Committee reference B/20
Draft for approval 69/239

ISBN 580 05523 X

Amendments issued since publication

Amd. No.	Date of issue	Comments
8859	November 1995	Indicated by a sideline in the margin

Contents

Foreword

BS 449 was first issued in 1932 and was revised in December 1935, July 1937, July 1948 and May 1959. It was reset and reprinted in February 1965 incorporating amendments Nos. 1 to 5 and reprinted in October 1967 incorporating amendments Nos. 6 and 7.

When a programme of Codes of Practice for Buildings was drawn up in 1942 under the aegis of the Ministry of Works, a Code of Practice for the structural use of steel in buildings was included in a series for all types of building construction: this was later (1948) issued as CP 113. Much of the information given in BS 449 and in CP 113 was the same and with the formation of the Codes of Practice Council within BSI it was decided that the two documents should be amalgamated and issued as a single publication under the main reference BS 449.

Part 2 of this standard has been prepared, in accordance with the change to the metric system in the construction industry, giving values in terms of SI units. For further information on SI units, reference should be made to BS 3763, “International System (SI) units” and PD 5686, “The use of SI units”.

The values given in Part 2 represent the equivalents of the values in imperial units in BS 449:1959, rounded to convenient numbers. Although the values are not exact equivalents of the imperial ones, Part 2 does not constitute a technical revision of the standard and is substantially similar in content to BS 449:1959.

Amendment Slip No. 8 (AMD 94) published 12th September, 1968 introduced into this standard the use of steels complying to BS 4360, “Weldable structural steels” superseding the requirements of BS 15, BS 968 and BS 2762. This amendment slip also noted the future insertion of grade 55 steel. Part 2 has now been expanded to include the use of this steel, and also includes an amendment to Subclause **3** *a*). Amendment Slip No. 6 (PD 5854) published 29th June, 1966 revised Subclause **3** *a*) and introduced into this standard, precautions against brittle fracture. This subclause on brittle fracture has now been further extended to be in parity with BS 4360. Because of the extent of the amendments made necessary by the publication of BS 4360 it has been found more practical to reissue the imperial edition of this standard incorporating these amendments rather than to publish a further amendment slip.

For the purpose of compliance with this standard designers may use the values given in either BS 449:1959 or this part, provided that one set of values is used consistently throughout the design of each part of a structure.

A revision of this standard is currently in hand and the revised standard will be written in terms of SI units only.

Users of this British Standard should satisfy themselves that effective compliance is secured with local bye-laws and regulations and, for insurance purposes, with any requirements of insurance companies.

The attention of users is also called to the importance of making provision, where necessary, for water, gas, electricity and other services, having particular regard to condition 5 in Clause **21** for cased beams and to condition 4 in Subclause **30** *b*) for cased struts.

This standard reflects the revised increased yield strength of grade 43 steels included in BS 4360.

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people, for whose guidance it has been prepared, and that construction and supervision is carried out by capable and experienced organizations.

Economy in design

This British Standard stipulates limits of stress and gives rules for design, with the twofold purpose of ensuring normal safety and economy in the use of structural steel. While the stresses and other requirements are to be regarded as limiting values, the purpose in design should be to reach these limits in as many parts of the structure as possible and to adopt a layout such that maximum structural efficiency is attained for a minimum use of steel. Careful consideration should therefore be given to the semi-rigid basis and fully rigid basis of design.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 78, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

1 Scope

This British Standard relates primarily to the use in building of hot rolled steel sections and plates, and normalized tubular shapes.

The use in building of cold formed sections in light gauge plate, sheet and strip steel 6 mm thick and under is dealt with in Addendum No. 1 to BS 449-2:1969.

The provisions of this standard are not deemed to apply to transmission towers and farm buildings; nor to structures which are designed on an experimental basis, except in so far as provided by Clause **9** *c*) and Appendix A.

Chapter 1. Definitions

2 Definitions

For the purposes of this British Standard the following definitions apply:

beam or girder

any structural member which supports load primarily by its internal resistance to bending

dead load

the weight of all permanent construction

effective lateral restraint

restraint which will produce sufficient resistance in a plane perpendicular to the plane of bending to restrain a loaded beam from buckling to either side at the point of application of restraint

engineer

the person responsible for the design and satisfactory completion of the structure, as covered by this specification

filler joists

rolled steel I-beams or other suitable flanged sections used in combination with structural concrete and forming the skeleton of a floor or roof slab

foundation

that part of the building the function of which is to distribute loading direct to the ground. It may include any retaining or other wall, based upon the ground, of sufficient strength and stability to carry its own weight together with the loads and forces imposed upon it

high strength friction grip bolts

high strength friction grip bolts are bolts of high tensile steel, used in conjunction with high strength nuts and hardened steel washers, which are tightened to a pre-determined shank tension in order that the clamping action thus afforded will transfer loads in the connected members by friction between the parts in contact and not by shear or bearing in the bolts (See footnote to Clause **48**.)

imposed load

in respect of a building: all loads other than the dead load

load factor

the numerical value by which the load which would cause failure of the structure is divided to give the permissible working load on the structure

panel wall

a wall built between pillars, stanchions or other members and supported by the steel framework or foundations

partition

an internal vertical structure which is employed solely for the purpose of subdividing any storey of a building into sections, and which supports no load other than its own weight

strut

a steel pillar, stanchion, column or other compression member

welding terms

except as otherwise defined in this British Standard, the terms used in the welding clauses have the meanings given in BS 499, "Welding terms and symbols"

wheel loads

the static weights imposed by the wheels when the appliance of which the wheels form part is fully loaded

yield stress

the yield stress in tension

Chapter 2. Materials

3 Structural steel and electrodes

a) **Structural steel**[1)]

i) All structural steel used in building coming within the purview of this British Standard shall, before fabrication, comply with grade 43, 50 or 55 of BS 4360[2)] and with such additional requirements as may be given in Subclauses ii) to vi) below.

ii) As a precaution against brittle fracture, steel used for elements which are subject to tensile stresses in service due to applied axial load or moment, shall comply with the following requirements:

1) Sections, plates, wide flats, flats and bars in elements which are subject to tensile stresses exceeding 60 N/mm^2 at welded locations, may be used up to the following thickness:

Grade	*Section, plates flats and bars*	*Wide flats*
43A	25 mm	25 mm
43B	30 mm	30 mm
43C	60 mm	50 mm
43D	100 mm	50 mm
50A	20 mm	20 mm
50B	25 mm	25 mm
50C	45 mm	45 mm
50D	100 mm	50 mm

For grades 43B and 50B, option B.39 of BS 4360[2)] shall be invoked when the steel is ordered.

2) Sections, plates, wide flats and bars in other tension elements may be used up to double the above thicknesses subject to a maximum thickness of 100 mm generally or 50 mm for wide flats.

3) Hot rolled hollow sections of grades 43C and 50C may be used up to a maximum thickness of 16 mm for rectangular hollow sections or 40 mm for circular hollow sections.

4) Steel of grade 55C may be used up to the following thicknesses:

Sections, flats and bars	19 mm
Plates, wide flats and circular hollow sections	25 mm
Rectangular hollow sections	16 mm

NOTE For members containing unreamed punched holes, see also Clause **59** *d*).

iii) The requirements in Subclause ii) above apply to the general use of structural steelwork in the United Kingdom, but further consideration is to be given to this aspect by the designer in the following circumstances:

1) Critical conditions where a single failure would result in the collapse of a major part of the structure.

2) Positions where complicated details are involved which may lead to high restraint and difficulties in inspection or testing or both.

3) Dynamic or shock loading producing tension.

4) Structures in which important elements of the steelwork are used in circumstances where the temperature may be below – 7 °C.

iv) Under any of the conditions in Subclause iii) above, one or more of the following safeguards additional to ii) above may be advisable:

1) Impact test requirements to be imposed on materials thinner than those specified in Subclause ii) above.

2) The impact test requirement to be increased.

3) Stress relieving to be carried out on the material.

v) In addition to the requirements of Subclause ii) above, where ambient temperatures below – 7 °C occur in service, the steel used shall have a minimum Charpy V-notch impact test value at the lowest ambient temperature of not less than the following:

grade 43	0.4t Joules
grade 50	0.5t Joules
grade 55	0.6t Joules

where t is the thickness in mm.

For steelwork used in external conditions in the UK the ambient temperature may be taken as – 15 °C.

[1)] A new specification for rivet steel, BS ...., "Steel rivet bars for the manufacture of structural and general engineering rivets" is in course of preparation.

[2)] BS 4360, "Weldable structural steels".

vi) For any conditions dealt with in Subclauses ii) to v) above, the impact test pieces shall be cut from the sections as specified in BS 4360[3)].

b) **High tensile steel rivets.** Where high tensile steel is used for rivets, steps shall be taken to ensure that the rivets are so manufactured that they can be driven and the heads formed satisfactorily; and that the physical properties of the steel are not impaired.

c) **Electrodes.** Mild steel electrodes and high tensile steel electrodes shall comply with the requirements of BS 639, "Covered electrodes for the manual metal-arc welding of mild steel and medium-tensile steel".

d) **Steel for bolts and nuts**

Bolts and nuts shall comply with one of the following specifications:

BS 3692.	ISO metric precision hexagon bolts, screws and nuts.
BS 4190.	ISO metric black hexagon bolts, screws and nuts.
BS 4933.	ISO metric black cup and countersunk head bolts and screws with hexagon nuts.

e) **Washers.** Plain washers shall be made of steel.

Taper or other specially shaped washers shall be made of steel or malleable cast iron. (See BS 4320, "Metal washers for general engineering purposes. Metric series.)".

4 Other materials

Other materials used in the structure in association with steelwork shall conform to any bye-laws or regulations to which the building has to conform.

Where an appropriate British Standard for a particular material exists the material shall also comply with that British Standard, except where it may conflict with, or differ from, the requirements of any relevant bye-law or regulation.

5 Standard dimensions

The dimensions of all structural rolled shapes, and the form, weight, tolerance, etc., of all rolled shapes and other members used in any structure shall, whenever possible, conform to the latest appropriate standards of the British Standards Institution.

The dimensions or form and the weight, tolerances, etc., of all rivets, bolts, nuts, studs, etc., shall conform to the requirements of the latest appropriate standards of the British Standards Institution.

Chapter 3. Loads

6 Dead loads and imposed loads

Reference should be made to CP 3:Chapter V, "Loading":

1) for recommendations on the determination of the dead load; and

2) for recommended values of imposed loads on floors, corridors, balconies, stairs and landings, parapets and balustrades; of snow and other loads; and of wind loads on buildings and unclad structures (including sheeted towers and chimneys).

7 Dynamic loads

Where loads arising from machinery, runways, cranes and other plant producing dynamic effects are supported by or communicated to the framework, allowance shall be made for these dynamic effects, including impact, by increasing the deadweight values by an adequate percentage.

In order to ensure due economy in design the Engineer shall ascertain as accurately as possible the appropriate dynamic increase for all members affected.

For crane gantry girders the following allowances shall be deemed to cover all forces set up by vibration, shock from slipping of slings, kinetic action of acceleration and retardation, impact of wheel loads and skew loads due to travelling:

1) For loads acting vertically, the maximum static wheel loads shall be increased by the following percentages:

for electric overhead cranes	25 per cent
for hand-operated cranes	10 per cent

Values for cranes for loading class Q3 and Q4 as defined in BS 2573-1 shall be established in consultation with the crane manufacturer.

2) The horizontal force acting transverse to the rails shall be taken as a percentage of the combined weight of the crab and the load lifted as follows:

for electric overhead cranes	10 per cent
for hand-operated cranes	5 per cent

This force shall be taken into account when considering the lateral rigidity of the rails and their fastenings. Provided that both gantry girders supporting the crane are similar, this horizontal force may be assumed to be shared equally between the two supporting girders.

[3)] BS 4360, "Weldable structural steels".

3) Horizontal forces acting along the rails shall be taken as a percentage of the static wheel loads which can occur on the rails, as follows:

for overhead cranes, either

electric or hand-operated 5 per cent

4) For gantry girders intended to carry cranes of class Q3 and Q4 as defined in BS 2573-1, two equal and opposite horizontal forces acting transverse to the rail, one at each end of the wheelbase, of the magnitude specified in Clause **3.1.5.2** of BS 2573-1.

The forces specified in 2), 3) or 4) above shall be considered as acting at the rail level and being appropriately transmitted to the supporting systems.

Gantry girders and their supporting structures shall be designed on the assumption that the horizontal forces 2), 3) and 4) are alternatives which do not act at the same time but that each of them may act at the same time as the vertical load.

An increase of 10 per cent on the allowable stresses specified in this standard shall be allowable for the combination of loadings 1) and 2), 3) or 4) above in respect of the design of the gantry girders and supporting structures. This increase is not however in addition to that permitted in Clause **13**.

In special cases, e.g. charging machines, and where more than one crane is in use on the gantry and where high speeds are attained, the above allowances should be reconsidered.

For cranes on outdoor gantries the wind loads on the gantry and supporting structure shall be obtained from:

a) BS 2573-1, for cranes in the working condition;

b) CP 3:Chapter V-2, for cranes which are not working.

Where a structure or member is subject to loads from two or more cranes, the crane loads shall be taken as the maximum vertical and horizontal loads acting simultaneously. Loading on steel over-head runway beams shall be as specified in BS 2853.

NOTE For factors applicable to the allowable working stresses, and detailed design under fatigue conditions, see BS 2573-1.

8 Temperature range

Where, in the design and erection of a building, it is necessary to take account of a change of temperature, it shall be assumed that the average temperature of the metal in the structure in Great Britain and Northern Ireland does not vary over a range greater than from – 7° C to + 50 °C. The design range, however, depends on the location, type and purpose of the building. Special consideration will be necessary for structures in special conditions and in localities abroad subject to different temperature ranges.

Chapter 4. Design and details of construction

A. General

9 Steel framework

a) Any part of the structure shall be capable of sustaining the most adverse combination of static and dynamic forces which may reasonably be expected from dead loads and all imposed loads, including snow and wind loads, referred to in Chapter 3, without the allowable stresses specified in Chapter 4 being exceeded. The design and details of parts and components shall be compatible so that the required overall stability is achieved.

NOTE This is best ensured by making one person responsible for this compatibility when some or all of the design and details are not made by the same designer.

b) The following methods may be employed in the design of the steel framework:

1) *Simple design.* This method applies to structures in which the end connections between members are such that they will not develop restraint moments adversely affecting the members and the structure as a whole and in which the structure may, for the purposes of design, be assumed to be pin-jointed.

This method of design involves the following assumptions:

Beams are simply supported.

All connections of beams, girders or trusses are proportioned to resist the reaction shear forces applied at the appropriate eccentricity.

Members in compression are subjected to loads applied at the appropriate eccentricities (see Clause **34** for stanchions), with the effective lengths given in Clauses **30** and **31**.

Members in tension are subjected to longitudinal loads applied over the net area of the section, as specified in Clause **42**.

2) *Semi-rigid design.* This method, as compared with the simple design method, permits a reduction in the maximum bending moment in beams suitably connected to their supports, so as to provide a degree of direction fixity and, in the case of triangulated frames, it permits account being taken of the rigidity of the connections and the moment of interaction of members. In cases where this method of design is employed, calculations based on general or particular experimental evidence shall be made to show that the stresses in any part of the structure are not in excess of those laid down in this British Standard.

An allowance may be made for the inter-restraint of the connection between a beam and a column by a moment not exceeding 10 per cent of the free-moment applied to the beam, assuming this to be simply supported, provided:

A) The beams and columns are designed by the general rules for members of a simply supported frame.

B) The beams are designed for the maximum net moment with due allowance for any difference in the restraint moment at each end.

C) Each column is designed to resist the algebraic sum of the restraint moments from the beams at the same level on each side of the column, in addition to moments due to eccentricity of connection (see Clause **34**).

D) The assumed end restraint moment need not however be taken as 10 per cent of the free-moment for all beams, provided that the same value of the restraint moment is used in the design of the column and the beam at each connection.

E) The column is fully encased in concrete in accordance with Clause **30** *b*) and the beam to column connection includes a top cleat.

3) *Fully rigid design.* This method, as compared with the method for simple and semi-rigid design, will give the greatest rigidity and economy in weight of steel used when applied in appropriate cases. For this purpose the design shall be carried out in accordance with accurate methods of elastic analysis and to the limiting stresses permitted in this British Standard. Alternatively, it shall be based on the principles of plastic design so as to provide a load factor of not less than 1.7 when considering only gravity loading and 1.36 when considering the combined effects of wind loads and gravity loading, and with deflections under working loads not in excess of the limits implied in this British Standard.

Due consideration shall also be given to the possibility of member instability and frame instability, both for elastic analysis of rigid-jointed frames and also when plastic design is used. The notional horizontal forces specified in Subclause **10** *c*) shall be included in the frame stability check.

c) **Experimental basis.** Where, by reason of the unconventional nature of the construction, calculation is not practicable, or where the methods of design given in *b*) above are inapplicable or inappropriate, loading tests shall be made, in accordance with the procedure set out in Appendix A, to ensure that the construction has:

1) adequate strength to sustain a total load equal to twice the sum of the dead load and the imposed load, and

2) adequate stiffness to resist, without excessive deflection, a total load equal to the sum of the dead load and 1½ times the imposed load and, in the case of a beam, to satisfy Clause **15**.

d) **Curved members and bends.** The design of curved or bent members shall be given special consideration. Allowance shall be made for any thinning of the bent part which may be caused by bending the member.

10 Resistance to horizontal forces

a) **General.**

i) When considering the effect of wind pressure on buildings, due allowance shall be made for the resistance and stiffening effects of floors, roofs and walls.

ii) When the floors, roof and walls are incapable of transmitting the horizontal forces to the foundation, the necessary steel framework shall be provided to transmit the forces to the foundations. This framework may be in the form of triangulated bracing to members, portal construction or cantilevers which shall comprise all members necessary effectively to transmit the forces to the foundations.

Resistance to the wind forces on the sides and roof of a building shall be provided by horizontal or inclined bracing or beam systems designed to transmit the wind loads direct to the foundations or to intermediate transverse frames or to end frames, and those frames shall be designed to transmit the loads to the foundations.

Resistance to the wind forces on the ends and roof of a building shall likewise be provided by horizontal or inclined bracing or beam systems designed to transmit the wind loads to the side framing, and the side framing shall be braced to transmit these loads to the foundations, or, alternatively, the wind forces on the ends may be taken direct to the foundations by vertical cantilevers.

In buildings where high-speed travelling cranes are supported by the structure or where a building or structure may be otherwise subject to vibration or sway, triangulated bracing or especially rigid portal systems shall be provided to reduce the vibration or sway to a suitable minimum.

b) **Stability.** The stability of the structure as a whole, or of any part of it shall be investigated, and weight or anchorage shall be provided so that the least restoring moment, including anchorage, shall be not less than the sum of 1.4 times the maximum overturning moment due to dead loads and 1.6 times the maximum overturning moment due to imposed loads.

When considering wind loads, the restoring moment shall not be less than 1.4 times the overturning moment due to dead loads and wind loads, nor less than 1.2 times the overturning moment due to the combined effects of dead, imposed and wind loads.

To ensure stability at all times account shall be taken of probable variations in dead load during construction, repair or other temporary measures.

In complying with the requirements of this clause it is necessary to ascertain that the resulting pressures and shear forces to be communicated by the foundations to the supporting soil would not produce failure.

All parts of the structure which have been designed for their dead, imposed and wind loads to the allowable stresses in this British Standard shall be deemed to be adequately covered for this margin of stability provided that no stress reversal takes place in the part when the loads contributing to the overturning moment are increased by the factors specified above whilst the loads contributing to the restoring moment remain unfactored.

Where such stress reversal occurs, the resulting stresses may exceed the allowable stresses in the relevant clauses of Chapter 4 of this standard by 40 per cent. This increase is not, however, in addition to that permitted in Clause **13** or in Clause **40**.

c) **Sway-stability.** All structures including portions between expansion joints shall be adequately strong and stiff to resist sway. To ensure adequate strength, in addition to designing for applied horizontal loads, a separate check shall be carried out for notional horizontal forces which can arise due to practical imperfections such as lack of verticality.

These notional horizontal forces shall be applied at each roof and floor level or their equivalent and shall be taken as equal to 0.5 per cent of the sum of the dead and imposed gravity loads applied at that level, but not less than 1.0 per cent of the dead load.

The notional horizontal forces shall be taken as acting simultaneously with the vertical loads. They shall be assumed to act in any one horizontal direction at a time.

Notional horizontal forces shall not be applied when:

considering stability against overturning.

applying wind loads or other horizontal loads.

considering temperature effects.

determining horizontal loads on foundations.

Whatever system is used to resist horizontal forces [see Subclause **10** *a*)], reversal of loading shall be accommodated. Where floors, walls or roofs are used to provide sway stability, they shall have adequate strength and be so secured to the structural framework as to transmit all horizontal forces to points of sway resistance. Where such sway stability is to be provided by construction other than the steel framework, the need for such construction and the forces acting upon it shall be clearly stated.

11 Foundations

The foundations of a building or other structure shall be so designed as to limit the pressures on the sub-soil to safe bearing values and to ensure such rigidity and restraints as have been allowed for in the design of the super-structure, including resistance to all horizontal forces. Due regard shall be paid to the effects of temperature and seasonal changes generally.

The weight of foundations shall be such as to comply with Subclause **10** *b*).

12 Minimum thickness of metal

The following provisions shall apply, except in the case of members on which special protection against corrosion is provided:

1) **General.** Steel used for external construction exposed to the weather or other corrosive influences shall be not less than 8 mm thick; and in construction not so exposed, not less than 6 mm thick. These provisions do not apply to the webs of British Standard rolled steel joints and channels, or to packings.

2) **Sealed tubes or sealed box sections.** Sealed tubes or sealed box sections used for external construction exposed to the weather shall be not thinner than 4 mm and for construction not so exposed shall be not thinner than 3 mm.

13 Stresses due to wind forces

Unless otherwise stated, the allowable stresses specified in the relevant clauses of Chapter 4 of this standard may be exceeded by 25 per cent in cases where an increase in stress is solely due to wind forces, provided that the steel section shall be not less than that needed if the wind stresses were neglected.

This provision does not apply to grillage beams (see Clause **40**).

In the case of any combination of stresses arising from bending and axial loading, including the effects of wind, the values of p_c and p_{bc} in Clause **14** *a*) and p_t and p_{bt} in Subclause **14** *b*) shall be increased by 25 per cent in satisfying the relationship to unity, provided that Subclauses **14** *a*) and *b*) shall also apply in respect of any combination of stresses arising from bending and axial loading not due to wind.

14 Combined stresses

a) **Bending and axial compression.** Members subject to both axial compression and bending stresses shall be so proportioned that, at any point:

$$\frac{f_c}{p_c} + \frac{f_{bcx}}{p_{bcx}} + \frac{f_{bcy}}{p_{bcy}} \leq 1$$

where

f_c = the calculated average axial compressive stress.

p_c = the allowable compressive stress in axially loaded struts (see Table 17).

f_{bcx} and f_{bcy} = the compressive stresses due to bending about the respective rectangular axis.

p_{bcx} and p_{bcy} = the appropriate allowable compressive stresses for members subject to bending (see Clause **19**).

In cased struts for which allowance is made for the load carried by the concrete in accordance with Subclause **30** *b*), the ratio f_c/p_c in the above expression shall be replaced by the ratio of the calculated axial load on the strut to the allowable axial load determined from Subclause **30** *b*)

b) **Bending and axial tension.** Members subject to both axial tension and bending stress shall be so proportioned that, at any point:

$$\frac{f_t}{p_t} + \frac{f_{bt}}{p_{bt}} \leq 1$$

and $$\frac{f_{bcx}}{p_{bcx}} + \frac{f_{bcy}}{p_{bcy}} \leq 1$$

where

f_t = the calculated axial tensile stress.

p_t = the permissible axial tensile stress (see Clause **41**).

f_{bt} = the resultant tensile stress due to bending about both principal axes.

p_{bt} = the appropriate allowable tensile stress in bending (see Clause **19**).

p_{bcx} = the appropriate allowable compressive stress for members subject to bending about the major axis (see Clause **19**).

c) **Bending and shear.** The equivalent stress f_e due to bending and shear shall not exceed the values of p_e given in Table 1.

The equivalent stress f_e is obtained from the following formula:

$$f_e = \sqrt{(f_{bt}^2 + 3f_q^2)} \quad \text{or} \quad \sqrt{(f_{bc}^2 + 3f_q^2)}$$

in which f_{bc} or f_{bt}, and f_q are the numerical values of the co-existent bending and shear stresses.

In addition the following relationship shall be satisfied:

$$\left[\frac{f_{bc}}{p_o}\right]^2 + \left[\frac{f_q'}{p_q'}\right]^2 \leq 1.25$$

where

f_q' = the average shear stress in the web.

p_q' = the allowable shear stress specified in Subclause **23** *b*).

p_o = the allowable bending stress specified in Clause **20** item 2 b) iii).

d) **Combined bearing, bending and shear stresses.** Where a bearing stress is combined with tensile bending and shear stresses under the most unfavourable conditions of loading, the equivalent stress f_e, obtained from the following formulae, shall not exceed the values of p_e given in Table 1.

$$f_e = \sqrt{(f_{bt}^2 + f_b^2 + f_{bt}.f_b + 3f_q^2)}$$
$$\text{or } f_e = \sqrt{(f_{bc}^2 + f_b^2 - f_{bc}.f_b + 3f_q^2)}$$

in which f_{bt}, f_{bc}, f_q and f_b are the numerical values of the co-existent bending, shear and bearing stresses.

In addition the following relationship shall be satisfied:

$$\left[\frac{f_{bc}}{p_o}\right]^2 + \left[\frac{f_q'}{p_q'}\right]^2 + \frac{f_{cw}}{p_{cw}} \leq 1.25$$

Where f_q', p_q' and p_o are as defined in Subclause **14** *c*) and f_{cw} and p_{cw} are as defined in Subclause **28** *a*).

Table 1 — Allowable Equivalent Stress p_e

Form	Grade	Thickness	P_e
Sections, bars, plates, wide flats and hot rolled hollow sections	43	≤ 40 40 but ≤ 100	250 230
	50	≤ 63 63 but ≤ 100	320 295
	55	≤ 25	390

NOTE The increases permitted by clauses **7** and **13** do not apply to these values.

15 Deflection of beams

The maximum deflection of a beam shall be such as will not impair the strength and efficiency of the structure, lead to damage of the finishings or be unsightly.

Measures may be necessary to nullify the effects of the deflection due to dead load by cambering or by adjustment in the casing.

The maximum deflection due to loads other than the weight of the structural floors or roof, steelwork and casing, if any, shall not exceed 1/360 of the span.

16 Overhang of walls

Where a wall, or leaf of a cavity wall, is placed eccentrically upon the flange of a supporting steel beam, the beam and its connections shall be designed for torsion, unless the beam is encased in solid concrete and reinforced in combination with an adjoining slab in such a way as to prevent the beam deforming torsionally.

17 Sectional areas

a) **General.** The gross sectional area shall be taken as the area of the cross section as calculated from the specified size.

The net sectional area shall be taken as the gross sectional area less deductions for rivet holes, bolt holes and open holes, or other deductions specified in this standard.

In making deductions for rivet and bolt holes the diameter of the hole shall be assumed to be 2 mm in excess of the nominal diameter of the rivet or bolt unless otherwise specified. For countersunk rivets or bolts the appropriate addition shall be made to the diameter of the hole.

Except as required by the following paragraph, the areas to be deducted shall be the sum of the sectional areas of the maximum number of holes in any cross section at right angles to the direction of stress in the member.

For 1) all axially loaded tension members;

2) plate girders of grade 43 steel and with d_1/t greater than 85;

3) plate girders of grade 50 steel and with d_1/t greater than 75; and

4) plate girders of grade 55 steel and with d_1/t greater than 65

(where t = the thickness of the web, and d_1 is the depth defined in Subclause **23** *b* as *d* (i) for vertically stiffened webs without horizontal stiffeners)

the area to be deducted when the holes are staggered shall be that given above or, if greater, the sum of the sectional areas of all holes in any zig-zag line extending progressively across the member or part of the member, less $\frac{s^2t_1}{4G}$ for each gauge space in the chain of holes, where s = the staggered pitch, i.e. the distance, measured *parallel* to the direction of stress in the member, centre-to-centre of holes in consecutive lines.

t_1 = the thickness of the holed material.

G = the gauge, i.e. the distance measured *at right angles* to the direction of stress in the member, centre-to-centre of holes in consecutive lines.

For sections such as angles with holes in both legs the gauge shall be measured along the centre of the thickness of the angle.

NOTE In a built-up member where the chains of holes considered in individual parts do not correspond with the critical chain of holes for the member as a whole, the value of any rivets or bolts joining the parts between such chains of holes shall be taken into account in determining the strength of the member.

b) **Rivets.** The nominal diameter of a rivet shall be the diameter cold before driving.

The gross area of a rivet shall be the cross-sectional area of the rivet hole.

c) **Bolts and screwed tension rods.** The net sectional area of a bolt or screwed tension rod with ISO metric threads shall be taken as the tensile stress area of the threaded part or the cross-sectional area of the unthreaded part, whichever is the lesser.

NOTE The tensile stress areas of ISO metric bolts are given in BS 3692, BS 4190 and BS 4933

18 Separators and diaphragms

Where two or more rolled steel joists or channels are used side by side to form a girder, they shall be connected together at intervals of not more than 1·5 m except in the case of grillage beams encased in concrete, where it is only necessary to make suitable provision for maintaining correct spacing. Through bolts and separators may be used provided that in beams having a depth of 300 mm or more not less than 2 bolts in the depth are used with each separator. When loads are required to be carried from one beam to the other or are required to be distributed between the beams, diaphragms and their fastenings designed with sufficient stiffness to distribute the load shall be used.

When loads are required to be carried from one tube to another, or are required to be distributed between tubes, diaphragms (which may be tubular) and their fastenings, designed with sufficient stiffness to distribute the load between the tubes, shall be used.

B. Design of members subject to bending

19 Bending stresses (beams other than plate girders)

a) **Rolled I-Beams. Universal beams and columns and rolled channels.** The bending stress in the extreme fibres, calculated on the effective section, shall not exceed:

1) for parts in tension, the appropriate value of p_{bt} in Table 2, and

2) for parts in compression,

A) for bending about the major axis (except as specified in Case B) the lesser of the values of p_{bc} given in Table 2 and Table 3a, Table 3b or Table 3c, as appropriate;

B) for bending about the major axis in beams with the applied loading substantially concentrated within the middle fifth of the unrestrained length or in columns with moments arising from the eccentricities specified in Clause **34**, the lesser of the values of p_{bc} given in Table 2 and Table 4a, Table 4b or Table 4c as appropriate;

C) for bending about the minor axis, the appropriate value of p_{bc} in Table 2.

In Tables 3 and 4:

l = effective length of compression flange (see Clause **26**)

r_y = radius of gyration about the minor axis

D = overall depth of beam

T = mean thickness of flange
= area of horizontal portion of flange, divided by its width.

For rolled sections, the mean thickness is that given in the appropriate British Standards or other reference book.

Tables 3 and 4 apply to ordinary building structures with simple end connections such as end plates or cleats. Where the end connections provide less restraint, the effective length l obtained from Clause **26** shall be increased by 10 per cent.

b) **Compound beams.** The bending stress in compound beams having equal flanges of uniform cross section throughout shall not exceed the values of p_{bc} and p_{bt} specified in Subclause **19** *a*).

Where flanges are unequal or vary in sections between points of effective lateral restraint, the procedures specified in Clause **20** shall be applied.

c) **Angles and tees.** The bending stress in the leg when loaded with the flange or table in compression shall not exceed the appropriate value of p_{bt} in Table 2. When loaded with the leg in compression, the permissible compressive stress shall be calculated from Clause **20**, with $K_2 = -1.0$ and T = thickness of leg.

d) **Tubes and rectangular hollow sections.** The bending stresses in tubes and rectangular hollow sections, where the ratio of depth to breadth is not greater than 4, shall not exceed the appropriate values of p_{bt} and p_{bc} in Table 2.

e) **Castellated beams.** The bending stresses in tension or compression calculated from the net section properties with due allowance for the secondary vierendeel effects of shear at the openings and the local effects of point loads, if any, at any point in the length of the beam, shall not exceed the appropriate value of p_{bc} or p_{bt} in Table 2.

In addition the compressive stress in the extreme fibres, calculated neglecting secondary effects, shall not exceed the appropriate value of p_{bc} obtained as specified for plate girders in Clause **20**, except that the 20 per cent increase in the value of C_s shall not be applied to castellated beams.

Table 2 — Allowable stress p_{bc} or p_{bt} in bending
*(See also Clauses **19** and **20** and Tables 3 and 4)*

Form	Grade	Thickness of material	p_{bc} or p_{bt}
Sections, bars, plates, wide flats and hot rolled hollow sections. Compound beams composed of rolled sections plated, with thickness of plate. Double channel sections forming a symmetrical I-section which acts as an integral unit.	43	≤ 40 > 40 but ≤ 100	180 165
	50	≤ 63 > 63 but ≤ 100	230 215
	55	≤ 25	280
Plate girders with single or multiple webs	43	≤ 40 > 40 but ≤ 100	170 155
	50	≤ 63 > 63 but ≤ 100	215 200
	55	≤ 25	265
Slab bases	All steels		185

Table 3a — Allowable stress p_{bc} in bending (N/mm^2) for case A of Clause 19 *a*) 2) for grade 43 steel

l/r_y	D/T									
	5	10	15	20	25	30	35	40	45	50
40	180	180	180	180	180	180	180	180	180	180
45	180	180	180	180	180	180	180	180	180	180
50	180	180	180	180	180	180	180	180	180	180
55	180	180	180	178	176	175	174	174	173	173
60	180	180	176	172	170	169	168	167	167	166
65	180	180	172	167	164	163	162	161	160	160
70	180	177	167	162	159	157	156	155	154	154
75	180	174	163	157	154	151	150	149	148	147
80	180	171	159	153	148	146	144	143	142	141
85	180	168	156	148	143	140	138	137	136	135
90	180	165	152	144	139	135	133	131	130	129
95	180	162	148	140	134	130	127	125	124	123
100	180	160	145	136	129	125	122	119	118	117
105	180	157	142	132	125	120	116	114	112	111
110	180	155	139	128	120	115	111	108	106	105
115	178	152	136	124	116	110	106	103	101	99
120	177	150	133	120	112	106	101	98	96	95
130	174	146	127	113	104	97	94	91	89	88
140	171	142	121	107	97	92	88	85	83	81
150	168	138	116	100	92	87	82	79	77	75
160	166	134	111	96	88	82	77	74	72	70
170	163	130	106	92	84	77	73	69	67	65
180	161	126	102	89	80	73	69	65	63	60
190	158	123	97	85	76	70	65	61	59	56
200	156	119	95	82	73	66	62	58	55	53
210	154	116	92	79	70	63	58	55	52	50
220	151	113	90	77	67	61	56	52	49	47
230	149	110	87	74	65	58	53	49	47	44
240	147	107	85	72	62	56	51	47	44	42
250	145	104	83	69	60	53	48	45	42	40
260	143	101	80	67	58	51	46	43	40	38
270	141	98	78	65	56	49	45	41	38	36
280	139	96	76	63	54	48	43	39	37	35
290	137	94	75	61	52	46	41	38	35	33
300	135	93	73	60	51	44	40	36	34	32

Table 3b — Allowable stress p_{bc} in bending (N/mm^2) for case A of Clause 19 *a*) 2) for grade 50 steel

l/r_y	D/T									
	5	10	15	20	25	30	35	40	45	50
40	230	230	230	230	230	230	230	230	230	230
45	230	230	230	230	230	230	230	230	230	230
50	230	230	230	230	230	230	229	228	228	228
55	230	230	230	225	223	221	220	219	219	219
60	230	230	223	218	214	213	211	210	210	209
65	230	229	217	210	207	204	203	202	201	200
70	230	224	211	203	199	196	194	193	192	192
75	230	220	205	197	192	188	186	185	183	183
80	230	215	200	190	184	181	178	176	175	174
85	230	211	194	184	177	173	170	168	166	165
90	230	207	189	178	170	166	162	160	158	157
95	230	204	184	172	164	158	154	152	150	148
100	230	200	180	166	157	151	147	144	142	140
105	230	197	175	161	151	144	140	136	133	132
110	228	193	171	155	145	138	132	128	126	123
115	226	190	166	150	139	131	125	121	118	115
120	224	187	162	145	133	124	118	113	111	109
130	220	181	154	135	122	112	108	104	101	99
140	216	175	146	126	112	105	99	96	93	91
150	212	169	139	117	105	98	92	88	85	83
160	208	164	132	110	99	91	86	82	79	76
170	205	158	126	105	94	86	80	76	73	70
180	201	153	119	100	89	81	75	71	67	65
190	198	149	113	96	84	76	70	66	63	60
200	195	144	109	92	80	72	66	62	59	56
210	192	139	105	88	76	68	63	58	55	53
220	189	135	102	85	73	65	59	55	52	49
230	185	130	99	81	70	62	56	52	49	47
240	183	126	95	78	67	59	54	50	46	44
250	180	122	92	75	64	57	51	47	44	42
260	177	118	90	73	62	54	49	45	42	40
270	174	114	87	70	60	52	47	43	40	38
280	171	111	84	68	58	50	45	41	38	36
290	169	109	82	66	56	48	43	40	37	34
300	166	106	80	64	54	47	42	38	35	33

Table 3c — Allowable stress p_{bc} in bending (N/mm²) for case A of Clause 19 *a*) 2) for grade 55 steel

l/r_y	D/T									
	5	10	15	20	25	30	35	40	45	50
40	280	280	280	280	280	280	280	280	280	280
45	280	280	280	280	280	280	280	280	280	280
50	280	280	280	280	278	276	275	275	274	274
55	280	280	276	270	267	265	264	263	262	262
60	280	280	268	261	257	254	252	251	251	250
65	280	275	260	251	246	243	241	240	239	238
70	280	269	252	242	236	233	230	229	227	227
75	280	263	244	233	227	222	219	217	216	215
80	280	258	237	225	217	212	209	206	205	204
85	280	252	230	217	208	202	198	196	194	192
90	280	247	224	209	199	193	188	185	183	181
95	280	242	217	201	190	183	178	175	172	170
100	280	238	211	194	182	174	168	164	161	159
105	277	233	205	186	174	165	159	154	151	148
110	275	229	199	179	166	156	149	144	140	137
115	272	224	194	172	158	147	140	134	130	127
120	269	220	188	166	150	139	131	125	121	119
130	264	212	178	153	135	123	117	113	110	107
140	258	205	168	141	122	113	107	103	99	97
150	253	197	158	130	114	105	99	94	90	88
160	249	190	149	121	107	98	91	86	83	80
170	244	183	140	114	100	91	84	80	76	73
180	239	177	132	108	95	85	79	74	70	68
190	235	170	124	103	89	80	74	69	65	63
200	231	164	119	98	85	76	69	64	61	58
210	227	158	114	94	80	71	65	61	57	54
220	223	152	110	90	77	68	62	57	54	51
230	219	147	106	86	73	65	58	54	51	48
240	215	141	102	83	70	62	56	51	48	45
250	211	136	99	79	67	59	53	49	45	43
260	207	130	96	76	64	56	51	46	43	41
270	204	125	92	74	62	54	48	44	41	39
280	200	121	90	71	60	52	46	42	39	37
290	197	118	87	69	58	50	45	41	38	35
300	193	115	84	67	56	48	43	39	36	34

Table 4a — Allowable stress p_{bc} in bending (N/mm^2) for case B of Clause 19 *a*) 2) for grade 43 steel

l/r_y	D/T									
	5	10	15	20	25	30	35	40	45	50
40	180	180	180	180	180	180	180	180	180	180
45	180	180	180	180	180	180	180	180	180	180
50	180	180	180	180	180	180	180	180	180	180
55	180	180	180	180	180	180	180	180	180	180
60	180	180	180	180	180	180	179	178	178	178
65	180	180	180	178	176	175	174	173	172	172
70	180	180	179	174	171	170	168	168	167	167
75	180	180	175	170	167	165	163	162	162	161
80	180	180	172	166	162	160	158	157	156	156
85	180	179	168	162	158	155	153	152	151	150
90	180	176	165	158	154	151	149	147	146	145
95	180	174	162	155	150	146	144	142	141	140
100	180	172	159	151	146	142	139	137	136	135
105	180	170	156	148	142	137	135	132	131	130
110	180	168	154	144	138	133	130	128	126	125
115	180	166	151	141	134	129	126	123	121	120
120	180	164	148	138	131	125	121	119	116	115
130	180	160	144	132	124	118	113	110	107	105
140	180	156	139	126	117	110	105	101	98	96
150	179	153	134	121	111	103	97	94	92	91
160	177	149	130	116	105	97	93	89	87	85
170	175	146	126	111	99	93	88	85	82	80
180	173	143	122	106	95	89	84	80	77	75
190	171	140	118	101	92	85	80	76	73	71
200	169	137	114	97	88	82	76	72	69	67
210	167	134	111	94	85	78	73	69	66	63
220	165	132	107	92	82	75	70	66	62	60
230	163	129	104	89	80	72	67	63	59	57
240	161	126	101	87	77	70	64	60	57	54
250	159	124	97	85	75	67	62	57	54	51
260	158	121	95	82	72	65	59	55	52	49
270	156	119	94	80	70	63	57	53	50	47
280	154	116	92	78	68	61	55	51	48	45
290	152	114	90	76	66	59	53	49	46	43
300	151	112	88	74	64	57	51	47	44	41

Table 4b — Allowable stress p_{bc} in bending (N/mm^2) for case B of Clause 19 a) 2) for grade 50 steel

l/r_y	D/T									
	5	10	15	20	25	30	35	40	45	50
40	230	230	230	230	230	230	230	230	230	230
45	230	230	230	230	230	230	230	230	230	230
50	230	230	230	230	230	230	230	230	230	230
55	230	230	230	230	230	230	230	230	230	230
60	230	230	230	230	229	228	227	226	225	225
65	230	230	230	226	223	221	219	218	218	217
70	230	230	226	220	216	214	212	211	210	210
75	230	230	221	214	210	207	205	204	203	202
80	230	230	217	209	204	200	198	196	195	195
85	230	227	212	203	197	194	191	189	188	187
90	230	223	208	198	192	187	184	182	181	180
95	230	220	203	193	186	181	178	175	174	172
100	230	217	199	188	180	175	171	169	167	165
105	230	214	195	183	175	169	165	162	160	158
110	230	211	192	179	170	163	159	155	153	151
115	230	208	188	174	164	158	153	149	146	144
120	230	206	184	170	159	152	147	143	139	137
130	230	200	177	161	150	141	135	130	126	124
140	230	195	171	153	140	131	124	118	114	111
150	227	190	165	146	132	121	113	109	106	103
160	224	186	159	139	123	112	106	102	98	96
170	221	181	153	132	115	106	100	95	92	89
180	218	177	147	125	109	101	94	90	86	83
190	215	173	142	119	105	96	89	84	80	77
200	212	169	137	113	100	91	84	79	76	73
210	210	165	132	109	96	87	80	75	71	68
220	207	161	127	105	92	83	76	71	67	64
230	204	157	122	101	89	79	73	68	64	61
240	202	153	118	98	85	76	69	64	61	57
250	199	150	113	95	82	73	66	61	58	55
260	197	146	110	92	79	70	64	59	55	52
270	195	143	107	89	77	68	61	56	52	50
280	192	139	105	87	74	65	59	54	50	47
290	190	136	102	84	72	63	57	52	48	45
300	188	133	100	82	69	61	55	50	46	43

Table 4c — Allowable stress p_{bc} in bending (N/mm^2) for case B of Clause 19 *a*) 2) for grade 55 steel

l/r_y	D/T									
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
40	280	280	280	280	280	280	280	280	280	280
45	280	280	280	280	280	280	280	280	280	280
50	280	280	280	280	280	280	280	280	280	280
55	280	280	280	280	280	280	280	280	280	280
60	280	280	280	280	276	274	273	272	271	270
65	280	280	279	272	267	265	263	262	261	260
70	280	280	272	264	259	256	253	252	251	250
75	280	280	266	256	250	247	244	242	241	240
80	280	277	259	249	242	238	235	233	232	230
85	280	273	253	242	234	229	226	224	222	221
90	280	268	248	235	227	221	217	215	213	211
95	280	264	242	228	219	213	209	205	203	202
100	280	260	237	222	212	205	200	197	194	192
105	280	256	232	216	205	197	192	188	185	183
110	280	252	227	210	198	190	184	179	176	174
115	280	248	222	204	191	182	176	171	167	164
120	280	245	217	198	184	175	168	162	158	155
130	280	238	208	187	172	161	152	146	141	138
140	278	231	200	177	160	147	138	131	125	122
150	273	225	191	167	148	134	124	119	115	112
160	269	219	184	157	137	123	115	110	106	103
170	265	213	176	148	127	115	108	102	98	95
180	261	207	169	140	119	109	101	95	91	88
190	258	202	162	131	113	103	95	89	95	82
200	254	197	155	123	108	97	90	84	79	76
210	250	191	149	118	103	92	85	79	75	71
220	247	186	142	114	99	88	80	75	70	67
230	243	181	136	110	94	84	76	71	66	63
240	240	177	130	106	91	80	73	67	63	60
250	237	172	124	102	87	77	69	64	60	57
260	234	167	120	98	84	74	66	61	57	54
270	231	163	117	95	81	71	64	58	54	51
280	228	159	114	92	78	68	61	56	52	49
290	224	154	111	89	75	66	59	54	50	47
300	222	150	108	86	73	63	57	52	48	45

20 Bending stresses (plate girders)

The bending stress in the extreme fibres, calculated on the effective section, shall not exceed:

1) for parts in tension, the appropriate value of p_{bt} in Table 2; and

2) for parts in compression,

a) for bending about the minor axis, the appropriate value of p_{bc} in Table 2;

b) for bending about the major axis, the least of the following:

i) the value of p_{bc} given in Table 2

ii) k_o times the value of p_{bc} given in Table 8

where $k_o = (1 + y_c/y_t) / 2$ but $k_o \leq 1$

where y_c = distance from the neutral axis of the member to the extreme fibre in compression.

y_t = distance from the neutral axis of the member to the extreme fibre in tension.

iii) the value of p_o given by the following:

for grade 43: $p_o = 220 - 0.4\ d_2 / t$ N/mm^2

for grade 50: $p_o = 285 - 0.6\ d_2 /t$ N/mm^2

for grade 55: $p_o = 350 - 0.8\ d_2 / t$ N/mm^2

where d_2 = twice the clear depth of web from the compression flange to the neutral axis

and t = web

Table 8 gives values of p_{bc} corresponding to the critical stress C_s calculated as follows from the values of A and B given inTable 7 for appropriate values of D/T and l/r_y.

In Table 7:

l = effective length of compression flange (see Clause **26**)

r_y = radius of gyration about the minor axis of the gross section of the whole girder, at the point of maximum bending moment

D = overall depth of girder at the point of maximum bending moment

T = mean thickness of the horizontal portions of the flanges

= sum of areas of horizontal portions, divided by the sum of their widths.

Table 7 applies to ordinary building structures with simple end connections such as end plates or cleats. Where the end connections provide less restraint, the effective length l obtained from Clause **26** shall be increased by 10 per cent.

The values of C_s given below apply to all sections with a single web.

Case I. Where the flanges are equal in size:

$C_s = K_1\ A$ N/mm^2

Except that these values of C_s shall be increased by 20 per cent when:

T/t is not greater than 1.8

and B/T is not greater than 22

where B = overall width of girder at the point of maximum bending moment

NOTE This 20 per cent increase does not apply to castellated beams, see Clause **19** *e*).

In the above expression, K_1 is a coefficient which makes allowance for the reduction in thickness or breadth of flanges between points of effective lateral restraint and depends on the ratio N. The ratio N is the ratio of the flange area at the point of least bending moment to that at the point of greatest bending moment between adjacent points of effective lateral restraint. N refers *either* to the ratio of total area of both flanges *or* to the area of the compression flange only, whichever gives the smaller value of N.

Flanges should not be reduced in size to give a value of N lower than 0.2.

Values of K_1 or different values of N are given in Table 5. Values of A for different values of l/r_y and D/T are given in Table 7.

Case II. Where the flanges are unequal in size:

$C_s = K_1\ (A + K_2 B)\ (y_c/y_{min})$ N/mm^2

where K_2 is a coefficient to allow for inequality of tension and compression flanges and depends on the ratio M. The ratio M is the ratio of the moment of inertia of the compression flange alone to the sum of the moments of inertia of both flanges, each calculated about the minor axis of the member, at the point of maximum bending moment. Values of K_2 for different values of M are given in Table 6. Values of B for different values of l/r_y are given in Table 7.

y_{min} = the lesser of y_c and y_t.

Table 5 — Values of K_1

N	1.00	0.90	0.80	0.70	0.60	0.50	0.40	0.30	0.20
K_1	1.00	0.91	0.83	0.76	0.69	0.64	0.59	0.55	0.51

NOTE The values in Table 5 are obtained from: $K_1 = 1/(1.5\text{-}0.5N)^2$, but not greater than 1.0.

Table 6 — Values of K_2

M	1.00	0.90	0.80	0.70	0.60	0.50	0.40	0.30	0.20	0.10	0.00
K_2	0.50	0.40	0.30	0.20	0.10	0.00	–0.20	–0.40	–0.60	–0.80	–1.00

Table 7 — Values of A and B for calculating C_s

$$\text{where } A = \left(\frac{1675}{l/r_y}\right)^2 \sqrt{\left\{1 + \frac{1}{20}\cdot\left(\frac{lT}{r_y D}\right)^2\right\}}$$

$$B = \left(\frac{1675}{l/r_y}\right)^2$$

l/r_y	*A*															*B*
D/T	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40	2 630	2 353	2 187	2 081	2 009	1 958	1 921	1 862	1 830	1 810	1 797	1 781	1 773	1 764	1 761	1 754
45	2 226	1 965	1 808	1 706	1 637	1 587	1 551	1 494	1 461	1 442	1 429	1 413	1 405	1 396	1 392	1 385
50	1 929	1 683	1 534	1 436	1 369	1 321	1 286	1 229	1 198	1 178	1 165	1 150	1 142	1 133	1 129	1 122
55	1 701	1 470	1 328	1 235	1 170	1 123	1 089	1 034	1 002	983	970	955	947	938	934	927
60	1 522	1 304	1 169	1 079	1 017	972	938	884	854	835	822	807	799	790	786	779
65	1 377	1 172	1 043	957	897	854	821	768	738	719	707	692	683	675	671	664
70	1 258	1 064	941	859	801	759	727	676	646	627	615	600	592	583	580	573
75	1 158	974	857	778	723	682	651	601	571	553	541	526	518	510	506	499
80	1 074	898	787	711	658	618	588	539	510	492	480	466	457	449	445	438
85	1 001	834	727	655	603	565	536	488	460	442	430	415	407	399	395	388
90	938	778	676	607	557	520	491	445	417	400	388	373	365	357	353	346
95	882	730	632	565	517	481	454	408	381	364	352	338	330	322	318	311

Table 7 — Values of *A* and *B* for calculating C_s

l/r_y	*A*															*B*
D/T	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
100	833	687	593	529	482	447	421	376	350	333	321	307	299	291	287	281
110	750	616	529	469	425	393	368	325	300	283	272	258	251	243	239	231
120	682	558	477	421	380	350	326	286	261	246	235	221	213	206	202	195
130	626	510	435	383	344	315	293	255	231	216	205	192	184	177	173	166
140	578	470	400	351	315	287	266	229	207	192	182	169	161	154	150	143
150	537	436	370	324	290	264	243	209	187	173	163	150	143	135	132	125
160	502	407	345	301	268	244	225	191	171	157	147	135	128	120	116	110
170	471	382	322	281	250	227	208	177	157	143	134	122	115	107	104	97
180	444	359	303	264	234	212	195	164	145	132	123	111	104	97	93	87
190	420	339	286	248	221	199	182	153	135	122	113	102	95	88	84	78
200	398	321	271	235	208	188	172	144	126	114	105	94	87	80	77	70
210	379	305	257	223	197	178	162	135	118	106	98	87	81	74	70	64
220	361	291	245	212	187	169	154	128	111	100	92	81	75	68	65	58
230	345	278	233	202	179	161	146	121	105	94	86	76	70	63	60	53
240	330	266	223	193	170	153	139	115	100	89	82	71	65	59	55	49
250	317	255	214	185	163	146	133	110	95	85	77	67	61	55	51	45
260	304	245	205	177	156	140	128	105	91	80	73	64	58	51	48	42
270	293	236	197	170	150	135	122	101	86	77	70	60	55	48	45	38
280	282	227	190	164	145	130	118	97	83	73	66	57	52	45	42	34
290	272	219	183	158	139	125	113	93	79	70	64	55	49	43	40	33
300	263	211	177	153	134	120	109	89	76	67	61	52	47	41	38	31

Table 8 — Allowable compression stress p_{bc} in bending for different values of C_s
(but *see* Table 2)

C_s	p_{bc} for grade 43 steel		p_{bc} for grade 50 steel		p_{bc} for grade 55 steel
	Up to and including 40 mm	**Over 40 mm up to and including 100 mm**	**Up to and including 63 mm**	**Over 63 mm up to and including 100 mm**	**Up to and including 25 mm**
N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
10	6	6	6	6	6
20	11	11	11	11	11
30	16	16	16	16	17
40	21	21	22	21	22
50	26	26	27	27	27
60	31	30	32	31	32
70	35	35	37	36	37
80	40	39	41	41	42
90	44	43	46	45	47
100	48	47	51	50	52
110	52	51	55	54	57
120	56	54	59	58	62
130	59	58	64	62	66
140	63	61	68	66	71
150	66	64	72	70	75
160	69	67	76	74	79
170	72	70	79	77	84
180	75	73	83	81	88
190	78	75	86	84	92
200	81	78	90	87	96
210	83	80	93	90	100
220	85	82	96	93	103
230	88	84	99	96	107
240	90	86	102	99	111
250	92	88	105	102	114
260	94	89	108	104	117
270	96	91	110	106	121
280	97	93	113	109	124
290	100	95	117	112	129
300	102	96	120	115	133
310	104	97	123	118	137
320	105	98	126	121	141
330	106	99	129	124	145
340	107	99	132	125	149
350	107	100	135	127	153

Table 8 — Allowable compression stress p_{bc} in bending for different values of C_s
(but *see* Table 2)

C_s	p_{bc} for grade 43 steel		p_{bc} for grade 50 steel		p_{bc} for grade 55 steel
	Up to and including 40 mm	**Over 40 mm up to and including 100 mm**	**Up to and including 63 mm**	**Over 63 mm up to and including 100 mm**	**Up to and including 25 mm**
N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
360	108	101	136	128	156
370	109	101	137	129	159
380	109	102	138	130	162
390	110	102	139	131	165
400	111	104	140	132	166
420	112	106	142	133	169
440	115	109	144	135	172
460	117	112	146	136	175
480	120	114	147	137	177
500	122	116	149	139	179
520	125	119	150	141	181
540	127	121	151	144	183
560	130	123	154	146	185
580	132	125	156	149	186
600	134	127	159	152	188
620	136	129	162	154	189
640	138	131	164	156	191
660	140	133	167	159	192
680	142	135	169	161	194
700	144	136	171	163	197
720	146	138	174	165	199
740	147	140	176	167	202
760	149	141	178	169	205
780	151	143	180	171	207
800	152	144	182	173	210
850	156	148	187	178	216
900	160	151	192	182	221
950	163	155	196	186	226
1 000	166	156	200	190	231
1 050	169	158	204	193	236
1 100	171	160	207	197	240
1 150	173	161	211	200	245
1 200	175	163	214	203	248
1 250	176	164	217	206	252
1 300	177	165	220	208	256
1 400	180	165	226	213	262
1 500	180	165	230	215	269
1 600	180	165	230	215	274
1 700	180	165	230	215	279
1 800	180	165	230	215	280

21 Bending stresses (cased beams)

Beams and girders with equal flanges may be designed as cased beams when the following conditions are fulfilled:

1) The section is of single web and I-form or of double open channel form with the webs not less than 40 mm apart.

2) The beam is unpainted and is solidly encased in ordinary dense concrete, with 10 mm aggregate (unless solidity can be obtained with a larger aggregate), and of a works strength not less than 30 N/mm^2 at 28 days, when tested in accordance with BS 1881, "Methods of testing concrete".

3) The minimum width of solid casing is equal to b + 100 mm where b is the overall width of the steel flange or flanges in millimetres.

4) The surface and edges of the flanges of the beam have a concrete cover of not less than 50 mm.

5) The casing is effectively reinforced with wire complying with BS 4482, Hard drawn mild steel wire for the reinforcement of concrete'. The wire shall be of at least 5 mm diameter and the reinforcement shall be in the form of stirrups or binding at not more than 200 mm pitch, and so arranged as to pass through the centre of the covering to the edges and soffit of the lower flange. Alternatively, the casing may be reinforced with fabric complying with BS 4483, "Steel fabric for the reinforcement of concrete", or with bars complying with BS 4449, "Hot rolled steel bars for the reinforcement of concrete", provided in either case that the same requirements of diameter, spacing and positioning are met.

The compressive stress in bending shall not exceed the value of p_{bc} obtained from Clauses **19** and **20**; for this purpose the radius of gyration r_y may be taken as 0.2(b + 100 mm), and D/T as for the uncased section.

The stress shall not, however, exceed 1½ times that permitted for the uncased section.

See Clauses **29** and **40** for solidly encased filler joists and grillage beams respectively.

NOTE This clause does not apply to beams and girders having a depth greater than 1 000 mm or a width greater than 500 mm or to box sections.

22 Bearing stress

The calculated bearing stress on the net projected area of contact shall not exceed the values of p_b given in Table 9.

Table 9 — Allowable bearing stress p_b

Form	Grade	Thickness	p_b
		mm	N/mm^2
Sections, bars, plates, wide flats and hot rolled hollow sections	43	≤ 40	210
		> 40 ≤ 100	190
	50	≤ 63	260
		> 63 ≤ 100	245
	55	≤ 25	320

23 Shear stresses

a) **Maximum shear stress.** The maximum value f_q of the shear stress, having regard to the distribution of stresses in conformity with the elastic behaviour of the members in flexure, shall not exceed values of p_q given in Table 10.

In calculating the resistance of tubes to shear the total shear force resisted at any section shall be taken as the product of half the gross sectional area of the tube and the appropriate maximum shear stress in the table. Where there are holes in the section, calculations shall be made to show that the maximum shear stress given in the table is not exceeded.

Table 10 — Allowable maximum shear stress p_q

Form	Grade	Thickness	p_b
		mm	N/mm^2
Sections, bars, plates, wide flats and hot rolled hollow sections	43	≤ 40	125
		> 40 ≤ 100	115
	50	≤ 63	160
		> 63 ≤ 100	145
	55	≤ 25	195

b) **Average shear stress on webs of I-beams, plates[4] and hot rolled sections.** The average shear stress f'_q on the gross section of the web shall not exceed the values of p'_q given in Table 11 for unstiffened webs or, for stiffened webs, the values given in Table 12a, Table 12b or Table 12c, as appropriate. For stiffened webs over 40 mm thick for grade 43 steel or 63 mm thick for grade 50 steel, this stress shall not exceed the lesser of the values given by Table 12 and Table 11.

The gross section of the web shall be taken as

for rolled I-beams and channels: the depth of the beam multiplied by the web thickness.

for plate girders: the depth of the web plate multiplied by its thickness.

[4] The term "plates" denotes plates and wide flats.

Compliance with this subclause shall be deemed to satisfy the requirements of *a*) above.

For webs which have tongue plates or which are reinforced by additional plates [see Subclause **27** *a*) iv)], the maximum shear shall be calculated and the beam designed so as to satisfy both *a*) and *b*) above.

In Table 12a, Table 12b and Table 12c:

d = (i) *for vertically stiffened webs without horizontal stiffeners*, the clear distance between flange angles or, where there are no flange angles, the clear distance between flanges, ignoring fillets. Where tongue plates (Figure 1) having a thickness of not less than twice the thickness of the web plate are used, the depth d shall be taken as the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thicknesses of the tongue plates, whichever is the less.

(ii) *for vertically stiffened webs with horizontal stiffeners as described in Subclause* **28** *b*), the clear distance between the tension flange (angles, flange plate or tongue plate) and the horizontal stiffener.

t = the thickness of the web.

NOTE 1 For the minimum thickness of web plates and the design of web stiffeners, see Clauses **27** and **28**.

NOTE 2 The allowable stresses given in the tables of Clause **23** apply provided any reduction of the web cross section is due only to rivet holes, etc. Where large apertures are cut in the web a special analysis shall be made to ensure that the stresses laid down in this standard are not exceeded.

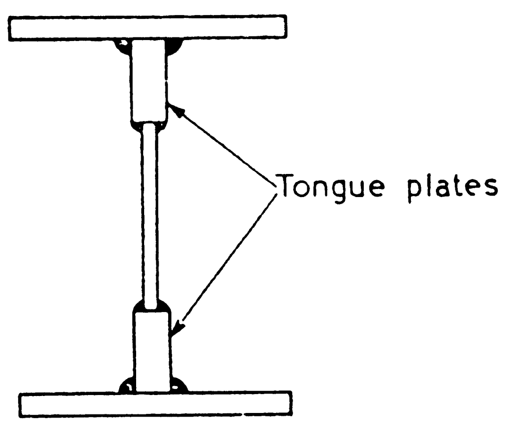


Figure 1 — Tongue plates

24 Effective span of beams

The effective span of a beam shall be taken as the length between the centres of the supports, except for the provision of 3 in Subclause **9** *b*) and where, under Clause **34**, the point of application of the reaction is taken as eccentric to the support, when it shall be permissible to take the effective span as the length between the assumed points of application of the reactions.

25 Maximum slenderness ratio of compression flanges

The ratio of the effective length l of the compression flange to the appropriate radius of gyration shall not exceed 300.

Table 11 — Allowable average shear stress p_q' in unstiffened webs

Form	Grade	Thickness	p_q'
		mm	N/mm^2
Sections, bars, plates, wide flats and hot rolled hollow sections	43	≤ 40	110
		> 40 ≤ 100	100
	50	≤ 63	140
		> 63 ≤ 100	130
	55	≤ 25	170

Table 12a — Allowable average shear stress p_q' in stiffened webs of grade 43 steel

d/t	Stress p_q' (N/mm^2) for different distances between stiffeners												
	0.3d	**0.4**d	**0.5**d	**0.6**d	**0.7**d	**0.8**d	**0.9**d	**1.0**d	**1.1**d	**1.2**d	**1.3**d	**1.4**d	**1.5**d
≤ 80	110	110	110	110	110	110	110	110	110	110	110	110	110
85	110	110	110	110	110	110	110	110	110	110	109	107	105
90	110	110	110	110	110	110	110	110	109	106	103	101	99
95	110	110	110	110	110	110	110	107	103	100	97	95	94
100	110	110	110	110	110	110	109	102	98	95	93	91	89
105	110	110	110	110	110	110	103	97	93	91	88	86	85
110	110	110	110	110	110	107	99	93	89	86	84	82	81
115	110	110	110	110	110	102	94	89	85	83	81	79	77
120	110	110	110	110	107	98	90	85	82	79	77	76	74
125	110	110	110	110	103	94	87	82	78	76	74	73	71
130	110	110	110	110	99	90	84	78	75	73	71	70	68
135	110	110	110	107	95	87	80	76	73	70	69	67	66
140	110	110	110	103	92	84	78	73	70	68	66	65	64
145	110	110	110	100	89	81	75	70	68	66	64	63	61
150	110	110	110	97	86	78	72	68	65	63	62	60	59
155	110	110	108	93	83	76	70	66	63	61	60	58	57
160	110	110	105	90	80	73	68	64	61	59	58	57	56
170	110	110	99	85	76	69	64	60	58	56	54	53	52
180	110	110	93	80	72	65	60	57	54	53	51	50	49
190	110	107	88	76	68	62	57						
200	110	102	84	72	64	59	54						
210	110	97	80	69	61	56							
220	110	93	76	66	59	53							
230	110	89	73	63	56								
240	110	85	70	60	54								
250	106	82	67	58	52								
260	102	78	65	56									
270	98	76	62	54									

NOTE For material over 40 mm thick, see also Subclause **23** *b*) and Table 11.

Table 12b — Allowable average shear stress p_q' in stiffened webs of grade 50 steel

d/t	Stress p_q' (N/mm^2) for different distances between stiffeners												
	0.3*d*	**0.4***d*	**0.5***d*	**0.6***d*	**0.7***d*	**0.8***d*	**0.9***d*	**1.0***d*	**1.1***d*	**1.2***d*	**1.3***d*	**1.4***d*	**1.5***d*
≤ 70	140	140	140	140	140	140	140	140	140	140	140	140	140
75	140	140	140	140	140	140	140	140	140	140	139	136	134
80	140	140	140	140	140	140	140	140	138	134	131	128	125
85	140	140	140	140	140	140	140	135	130	126	123	120	118
90	140	140	140	140	140	140	136	128	123	119	116	114	112
95	140	140	140	140	140	139	129	121	116	113	110	108	106
100	140	140	140	140	140	132	122	115	111	107	104	102	100
105	140	140	140	140	138	126	117	110	105	102	100	97	96
110	140	140	140	140	132	120	111	105	101	97	95	93	91
115	140	140	140	140	126	115	107	100	96	93	91	89	87
120	140	140	140	136	121	110	102	96	92	89	87	85	84
125	140	140	140	131	116	106	98	92	89	86	84	82	80
130	140	140	140	126	112	102	94	88	85	82	80	79	77
135	140	140	140	121	108	98	91	85	82	79	77	76	74
140	140	140	135	117	104	94	87	82	79	77	75	73	72
145	140	140	131	113	100	91	84	79	76	74	72	71	69
150	140	140	126	109	97	88	82	77	74	71	70	68	67
155	140	140	122	105	94	85	79	74	71	69	67	66	65
160	140	140	118	102	91	83	77	72	69	67	65	64	63
170	140	135	111	96	85	78	72	68	65	63	61	60	59
180	140	128	105	91	81	73	68	64	61	60	58	57	56
190	140	121	100	86	76	70	64						
200	140	115	95	82	73	66	61						
210	140	110	90	78	69	63							
220	136	105	86	74	66	60							
230	130	100	82	71	63								
240	125	96	79	68	61								
250	120	92	76	65	58								
260	115	88	73	63									
270	111	85	70	60									

NOTE For material over 63 mm thick, see also Subclause **23** *b*) and Table 11.

Table 12c — Allowable average shear stress p_q' in stiffened webs of grade 55 steel

d/t	Stress p_q' (N/mm²) for different distances between stiffeners												
	0.3d	**0.4**d	**0.5**d	**0.6**d	**0.7**d	**0.8**d	**0.9**d	**1.0**d	**1.1**d	**1.2**d	**1.3**d	**1.4**d	**1.5**d
≤ 65	170	170	170	170	170	170	170	170	170	170	170	170	170
70	170	170	170	170	170	170	170	170	170	170	165	162	159
75	170	170	170	170	170	170	170	170	164	158	154	151	148
80	170	170	170	170	170	170	170	159	153	149	145	142	139
85	170	170	170	170	170	170	160	150	144	140	136	133	131
90	170	170	170	170	170	163	151	142	136	132	129	126	124
95	170	170	170	170	169	154	143	134	129	125	122	119	117
100	170	170	170	170	161	147	136	127	123	119	116	113	111
105	170	170	170	170	153	140	129	121	117	113	110	108	106
110	170	170	170	165	146	133	123	116	112	108	105	103	101
115	170	170	170	157	140	127	118	111	107	103	101	99	97
120	170	170	170	151	134	122	113	106	102	99	97	94	93
125	170	170	168	145	129	117	109	102	98	95	93	91	89
130	170	170	162	139	124	113	104	98	94	91	89	87	86
135	170	170	156	134	119	109	101	94	91	88	86	84	82
140	170	170	150	129	115	105	97	91	88	85	83	81	79
145	170	170	145	125	111	101	94	88	85	82	80	78	77
150	170	170	140	121	107	98	91	85	82	79	77	76	74
155	170	165	136	117	104	95	88	82	79	77	75	73	72
160	170	159	131	113	101	92	85						
170	170	150	124	106	95	86	80						
180	170	142	117	101	89	81							
190	170	134	111	95	85	77							
200	166	127	105	91	81								
210	158	121	100	86	77								
220	151	116	95	82	73								
230	144	111	91	79									
240	138	106	88	75									
250	133	102	84	72									
260	128	98	81										
270	123	94	78										

26 Effective length of compression flanges for beams and girders

a) For simply supported beams and girders where no lateral restraint of the compression flange is provided but where each end of the beam is restrained against *torsion,* the effective length l of the compression flange to be used in Clauses **19**, **20** and **21**, shall be taken as follows:

1) with ends of compression flanges unrestrained against lateral bending (i.e. free to rotate in plan at the bearings)

$$l = \text{span}$$

2) with ends of compression flanges partially restrained against lateral bending (i.e. cleated flange connections)

$$l = 0.85 \times \text{span}$$

3) with ends of compression flanges fully restrained against lateral bending (i.e. not free to rotate in plan at the bearings)

$$l = 0.7 \times \text{span}$$

Restraint against *torsion* can be provided by web or flange cleats

or bearing stiffeners acting in conjunction with the bearing of the beam,

or lateral end frames or other external supports to the ends of the compression flanges (see Note below),

or their being built into walls.

Where the load is applied to the top flange of the beam and both the load and the flange are free to move laterally, the above values of the effective length shall be increased by 20 per cent.

Where the ends of the beam are not restrained against torsion other than by fixity or dead bearing of the bottom flange, the effective length shall be increased by the depth of the beam D for each end not so restrained.

NOTE The end restraint element shall be capable of safely resisting, in addition to wind and other applied external forces, a horizontal force acting at the bearing in a direction normal to the compression flange of the beam at the level of the centroid of the flange and having a value equal to not less than 2½ per cent of the maximum force occurring in the flange.

b) For beams which are provided with members having sufficient strength and stiffness to give effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in *a*) above, the effective length of the compression flange shall be taken as the maximum distance, centre to centre, of the restraint members or 1.2 times this length in the case of loads applied to the top flange of the beam, if the load and the flange are free to move laterally.

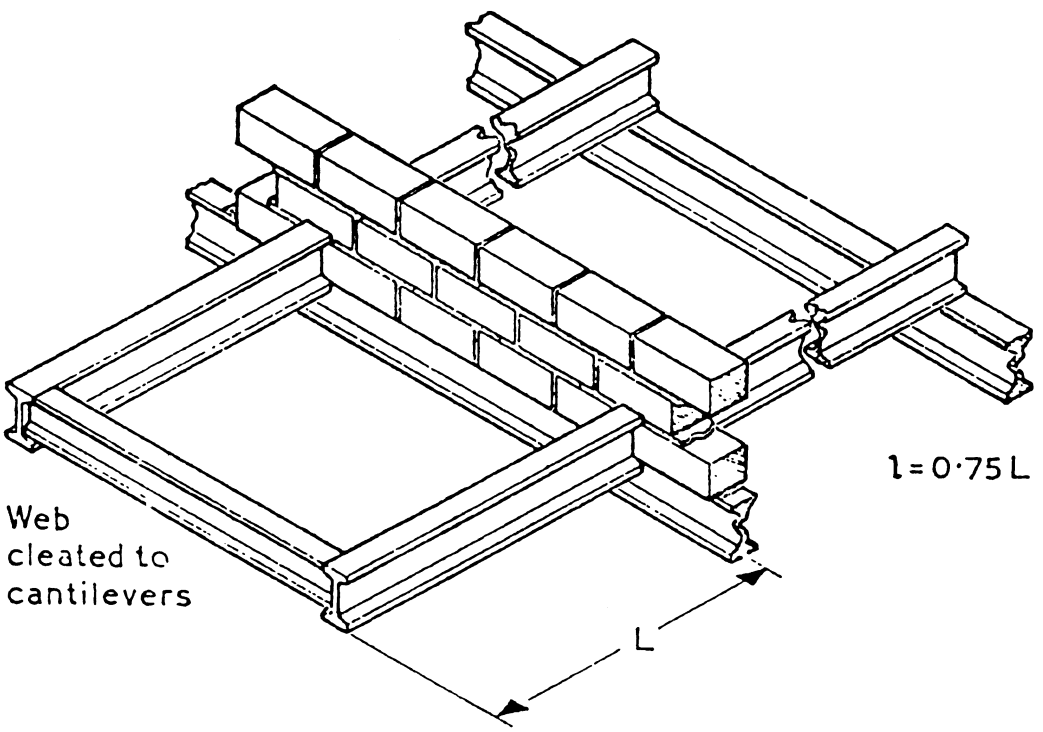


Figure 2a — Cantilever built-in at support, restrained against torsion at the end
(see 2 in Table 13)

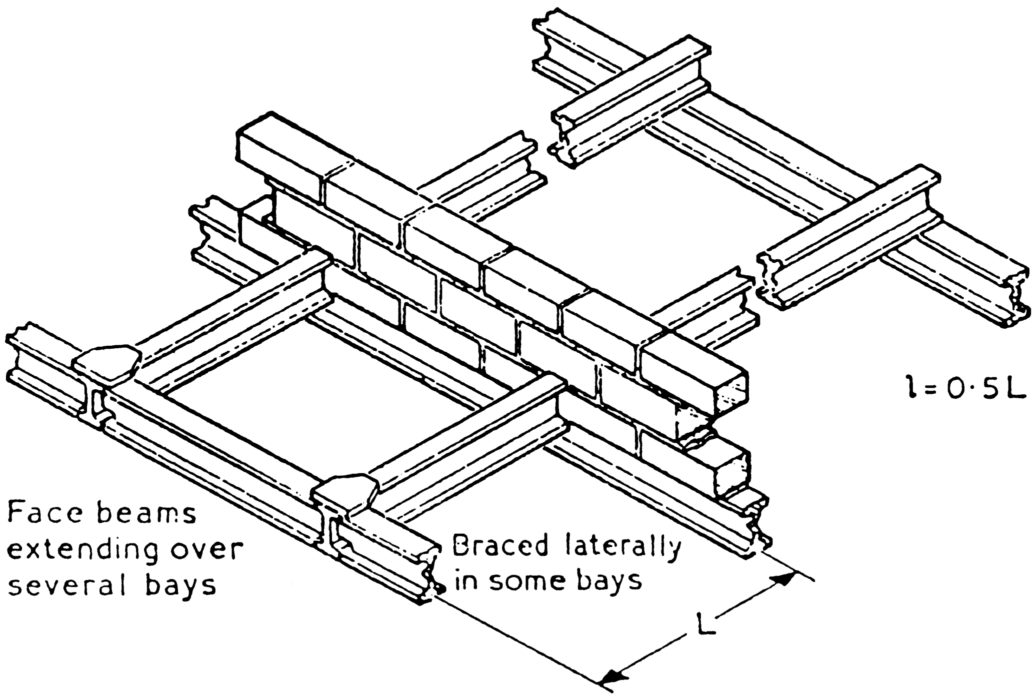


Figure 2b — Cantilever built-in at support, restrained laterally and torsionally at end
(see 3 in Table 13)

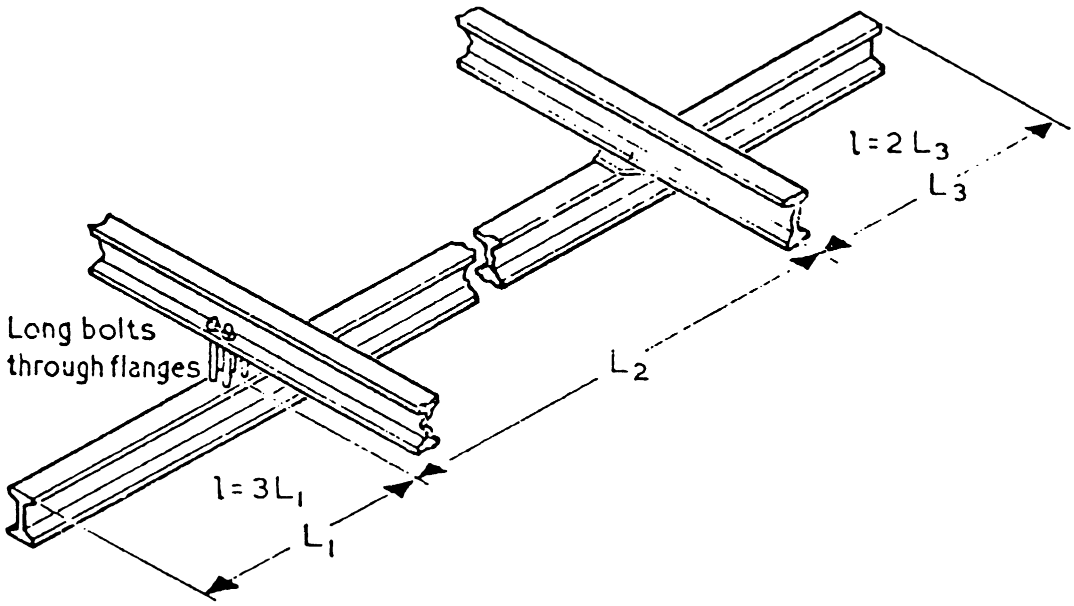


Figure 2c — Cantilever L_1 continuous at support, unrestrained against torsion at the support and unrestrained at the end
(see 4 in Table 13)

Figure 2d — Cantilever L_3 continuous at support, partially restrained against torsion at the support and unrestrained at the end
(see 5 in Table 13)

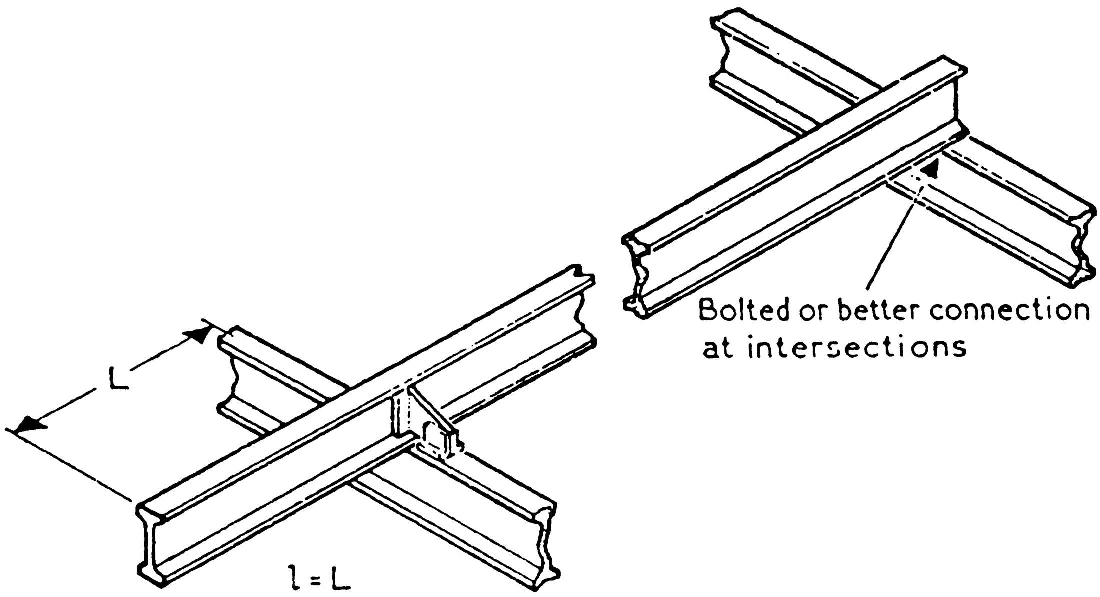


Figure 2e — Cantilever span continuous at support, fully restrained against torsion at the support and unrestrained at free end (see 6 in Table 13)

Table 13 — Effective lengths for cantilevers

Case	Conditions of restraint	Level of load	
		Normal	Top flange
1	Built-in at the support, free at the end	0.85 *L*	1.4 *L*
2	Built-in at the support, restrained against torsion at the end by contiguous construction (see Figure 2a)	0.75 *L*	0.75 *L*
3	Built-in at support, restrained against lateral deflection and torsion at the end (see Figure 2b)	0.5 *L*	0.5 *L*
4	Continuous at the support, unrestrained against torsion at the support and free at the end (see Figure 2c)	3 *L*	7.5 *L*
5	Continuous at the support with partial restraint against torsion of the support and free at the end (see Figure 2d)	2 *L*	5 *L*
6	Continuous at the support, restrained against torsion at the support and free at the end (see Figure 2e)	*L*	2.5 *L*

c) For cantilever beams of projecting length *L* the effective length *l* to be used in the subclause **19** *a*) shall be taken as given in Table 13.

Where the load is applied to the top flange of the cantilever and both the load and the flange are free to move laterally, the values for top flange loading shall be used.

Where the conditions of restraint at the support correspond to Cases 4, 5 and 6 above, the effective lengths obtained from 4, 5 and 6 shall be multiplied by 0.8 and 0.7 for degrees of restraint at the end corresponding to Cases 2 and 3 respectively.

d) Where beams support slab construction, the beam shall be deemed to be effectively restrained laterally if the frictional or positive connection of the slab to the beam is capable of resisting a lateral force of 2½ per cent of the maximum force in the compression flange of the beam, considered as distributed uniformly along the flange. Furthermore, the slab construction shall be capable of resisting this lateral force in lateral flexure and shear.

e) i) For beams which are provided with members giving effective lateral restraint of the compression flange at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2½ per cent of the maximum force in the compression flange taken as divided equally between the number of points at which the restraint members occur.

ii) In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2½ per cent of the maximum flange force in one beam only.

In the case of a series of latticed beams, girders or roof trusses which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2½ percent of the maximum force in the compression flange plus 1¼ per cent of this force for every member of the series other than the first up to a maximum total of 7½ per cent.

27 Beams with solid webs including plate girders

a) **Sectional properties.**

i) Solid web girders should preferably be proportioned on the basis of the moment of inertia of the gross cross section with the neutral axis taken at the centroid of that section, but it shall be permissible to use the net moment of inertia. In arriving at the maximum flexural stresses, the stresses calculated on the basis of the gross moment of inertia shall be increased in the ratio of gross area to effective area of the flange section. For this purpose the flange sectional area in riveted or bolted construction shall be taken to be that of the flange plate, flange angles and the portion of the web and side plates (if any) between the flange angles; in welded construction the flange sectional area shall be taken to be that of the flange plates plus that of the tongue plates (if any) up to a limit of eight times their thickness, which shall be not less than twice the thickness of the web.

ii) The effective sectional area of compression flanges shall be the gross area with the deductions for excessive width of plates as specified for compression members in Clause **32** and for open holes (including holes for pins and black bolts) occurring in a plane perpendicular to the direction of stress at the section being considered.

iii) The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in Subclause **17** *a*).

iv) The effective sectional area for parts in shear shall be taken as follows, subject to v) below:

For webs of plate girders. The product of the thickness of the web and the full depth of the web plate.

NOTE Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like, or where the proportion of the web included in the flange area is 25 per cent or more of the overall depth, the above approximation is not permissible and the maximum shear stress shall be computed.

For webs of rolled beams and channels. The product of the thickness of the web and the full depth of the section.

For other sections the maximum shear stress shall be computed from the whole area of the cross section, having regard to the distribution of flexural stresses.

v) Webs which have openings larger than those normally used for rivets or other fastenings require special consideration and the provisions of this clause are not applicable.

b) **Flanges**. In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than one third) and the number of flange plates shall be kept to a minimum.

In exposed situations where flange plates are used, at least one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is machined flush with the flange angles. Where two or more flange plates are used on one flange, tacking rivets shall be provided, if necessary, to comply with the requirements of Subclauses **51** *d*) and *e*).

Each flange plate shall be extended beyond its theoretical cut-off point, and the extension shall contain sufficient rivets or welds to develop in the plate the load calculated for the bending moment and girder section (taken to include the curtailed plate) at the theoretical cut-off point.

The outstand of flange plates, i.e. their projection beyond the outer line of connections to flange angles, to channel or joist flanges, or, in the case of welded constructions, their projection beyond the face of the web or tongue plate, shall not exceed the amounts given in Table 14 where *t* is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates tacked together.

Table 14 — Maximum outstand of flanges

Grade	Type of construction	Flange	Max. outstand
43	Plates with unstiffened edges	Compression Tension	$16t$ $20t$
	Channel sections or plates with continuously stiffened edges	Compression or tension	$20t$[a]
50	Plates with unstiffened edges	Compression Tension	$14t$ $20t$
	Channel sections or plates with continuously stiffened edges	Compression or tension	$20t$[a]
55	Plates with unstiffened edges	Compression Tension	$12t$ $20t$
	Channel sections or plates with continuously stiffened edges	Compression or tension	$20t$[a]

[a] To the innermost face of the stiffeners.

In the case of box girders, the thickness of any plate, or the aggregate thickness of two or more plates when these plates are tacked together to form the flange, shall be as required by Subclause **32** *a*).

c) **Flange splices.** Flange joints should preferably not be located at points of maximum stress.

Where cover plates are used their area shall be not less than 5 per cent in excess of the area of the flange element spliced; the centroid of their cross section shall coincide as nearly as possible with that of the cross section of the element spliced. There shall be enough rivets or welds on each side of the splice to develop the load in the element spliced plus 5 per cent. In welded construction flange plates shall be jointed by butt welds wherever possible. These butt welds shall develop the full strength of the plates.

d) **Connection of flanges to web.** The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the horizontal shear force combined with any vertical loads or reactions which are applied direct to the flange. In welded construction where the web is machined and in close contact with the flange before welding, it is permissible to design on the basis that such vertical loads are resisted by bearing between the flange and web.

e) **Dispersion of load through flange to web.** Where a load is applied direct to a flange, it shall be considered as dispersed uniformly through the flange to the connection of the flange to the web at an angle of 30° to the plane of the flange.

f) **Web plates.** i) *Minimum thickness.* The thickness t of the web plate shall be not less than the following:

1) For unstiffened webs:

$d_1/70$ for grade 43 steel.

$d_1/62$ for grade 50 steel.

$d_1/56$ for grade 55 steel.

where d_1 is the depth defined in Subclause **23** *b*) as *d*) i) for vertically stiffened webs without horizontal stiffeners.

2) For vertically stiffened webs:

1/180 of the smaller clear panel dimension and

$d_2/200$ for grade 43 steel.

$d_2/180$ for grade 50 steel.

$d_2/155$ for grade 55 steel.

3) For webs stiffened both vertically and horizontally with a horizontal stiffener at a distance from the compression flange equal to 2/5 of the distance from the compression flange to the neutral axis:

1/180 of the smaller dimension in each panel and

$d_2/250$ for grade 43 steel.

$d_2/225$ for grade 50 steel.

$d_2/190$ for grade 55 steel.

4) Where there is also a horizontal stiffener at the neutral axis of the girder:

1/180 of the smaller dimension in each panel and

$d_2/400$ for grade 43 steel.

$d_2/360$ for grade 50 steel.

$d_2/310$ for grade 55 steel.

In the above, d_2 is twice the clear distance from the compression flange angles, or plate, or tongue plate to the neutral axis.

In no case shall the greater clear dimension of a web panel exceed $270t$.

ii) *Riveted construction.* For girders in exposed situations and which have no flange plates, the top edge of the web plate shall be flush with or above the angles, as specified by the Engineer, and the bottom edge of the web plate shall be flush with or set back from the angles, as specified by the Engineer.

iii) *Welded construction.* The gap between web plates and flange plates shall be kept to a minimum, and for fillet welds shall not exceed 1 mm at any point before welding.

g) **Splices in webs.** Splices in the webs of plate girders and rolled sections used as beams shall be designed to resist the shears and moments in the web at the spliced section.

In riveted construction cover plates shall be provided on each side of the web.

h) **Side plates.** Where additional plates are provided to augment the strength of the web, they shall be placed on each side of the web and shall be equal in thickness. The proportion of shear force which these reinforcing plates shall be deemed to resist shall be limited by the amount of horizontal shear which they can transmit to the flanges through their fastenings, and such reinforcing plates and their fastenings shall be carried beyond the points at which they become theoretically necessary.

28 Web stiffeners

Web stiffeners shall be provided as follows:

a) **Load bearing stiffeners.**

i) *Rolled I-beams and channels.* For rolled I-beams and channels, load bearing stiffeners shall be provided at points of concentrated load (including points of support) where the stress f_{cw} calculated by dividing the concentrated load or reaction by tB exceeds p_{cw} where p_{cw} = the value of the allowable axial stress p_c for struts as given in Clause **30**, Table 17, for a slenderness ratio of 2.5 d_3/t.

where t = web thickness.

d_3 = clear depth of web between root fillets.

B = the length of the stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of the neutral axis, plus the thickness of flange plates at the bearing and the thickness of the seating angle (if any).

The stiff portion of a bearing is that length which cannot deform appreciably in bending, and shall not be taken as greater than half the depth of beam for simply supported beams and the full depth of the beam for beams continuous over a bearing [but see Subclause **14** *d*)].

The above expression for slenderness ratio assumes that the web acts as a strut fixed in position and direction at its ends and shall therefore only be used for calculating a safe buckling value if the following conditions are met:

1) The flange through which the load (or reaction) is applied is effectively restrained against lateral movement relative to the other flange.

2) Rotation of the loaded flange relative to the web is prevented.

If these conditions are not met the slenderness ratio shall be increased accordingly. Special consideration shall be given to cases where loads may act outside the plane of the web and so cause bending stresses in it.

ii) *Plate girders.* For plate girders load bearing stiffeners shall be provided at all points of support and at points of concentrated load exceeding the value specified in i) above or where the web would otherwise be overstressed [see Clause **22** and Subclause **14** *d*)].

Load bearing stiffeners shall be symmetrical about the web where possible.

iii) *All beams and girders.* Load bearing stiffeners shall be designed as struts, assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal, where available, to 20 times the web thickness. The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the working stress shall be in accordance with the appropriate allowable value for a strut, assuming an effective length equal to 0.7 of the length of the stiffener.

The outstanding legs of each pair of stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange clear of the root of the flange or flange angles, or clear of the welds, does not exceed the bearing stress specified in Clause **22**.

Load bearing stiffeners shall be provided with sufficient rivets or welds to transmit to the web the whole of the concentrated load.

The ends of load bearing stiffeners shall be fitted to provide a tight and uniform bearing upon the loaded flange unless welds designed to transmit the full reaction or load are provided between the flange and stiffener. At points of support this requirement shall apply at both flanges.

Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

For plate girders, where load bearing stiffeners at supports are the sole means of providing restraint against torsion [see Subclause **26** *a*)] the moment of inertia I of the stiffener shall be not less than:

$$\frac{D^3 T}{250} \cdot \frac{R}{W}$$

where I = moment of inertia of the stiffener about the centre line of the web plate.

D = overall depth of the girder.

T = maximum thickness of compression flange.

R = reaction on the bearing.

W = total load on the girder.

In addition, the bases of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to the horizontal force specified in the Note in Subclause **26** *a*).

b) **Intermediate stiffeners for plate girders.** To limit web buckling, intermediate stiffeners shall be provided as follows:

i) *Vertical stiffeners.* Vertical stiffeners shall be provided throughout the length of the girder at a distance apart not greater than $1\frac{1}{2}d_1$ [see also Subclause **23** *b*)] when the thickness of the web is less than $d_1/85$ for grade 43 steel or $d_1/75$ for grade 50 steel or $d_1/65$ for grade 55 steel where d_1 is the depth defined in Subclause **23** *b*) as d_1(i) for vertically stiffened webs without horizontal stiffeners.

These stiffeners shall be designed so that I is not less than

$$1.5 \times 10^{-4} \times \frac{{d_1}^3 \times t^3}{S^2}$$

where I = the moment of inertia in cm^4 of the complete stiffeners about the centre of the web.

S = the maximum permitted clear distance in mm between stiffeners for thickness t.

t = the minimum required thickness of web in mm.

d_1 = the depth, as defined above, in mm.

NOTE If the thickness of the web is made greater, or the spacing of the stiffeners made smaller than that required by this British Standard [see above and Subclause **23** *b*)], the moment of inertia of the stiffeners need not be correspondingly increased.

Intermediate vertical stiffeners may be joggled and may be single or in pairs placed one on each side of the web and shall extend from flange to flange, but need not have the ends fitted to provide a tight bearing on the flange.

ii) *Horizontal stiffeners.* Where horizontal stiffeners are used in addition to vertical stiffeners they shall be as follows:

One horizontal stiffener (single or double) shall be placed on the web at a distance from the compression flange equal to 2/5 of the distance from the compression flange to the neutral axis, when the thickness of the web is less than $d_2/200$ for grade 43 steel or $d_2/180$ for grade 50 steel or $d_2/155$ for grade 55 steel. This stiffener shall have a moment of inertia I not less than $4 \times 10^{-4} S_1 t^3$ where I and t are as defined in i) above, and S_1 is the actual distance between the stiffeners; d_2 is as defined in Subclause **27** *f*).

A second horizontal stiffener (single or double) shall be placed on the neutral axis of the girder when the thickness of the web is less than $d_2/250$ for grade 43 steel or $d_2/225$ for grade 50 steel or $d_2/190$ for grade 55 steel.

This stiffener shall have a moment of inertia I not less than $10^{-4} \times d_2 t^3$, where I and t are as defined in i) above and d_2 also is in mm.

Horizontal web stiffeners shall extend between vertical stiffeners but need not be continuous over them.

iii) *Outstand of stiffeners.* Unless the outer edge of each stiffener is continuously stiffened, the outstand of all stiffeners from the web shall be not more than the following:

For sections	$16t$ for grade 43 steel,
	$14t$ for grade 50 steel,
	$12t$ for grade 55 steel,
For flats	$12t$,

where t is the thickness of the section or flat.

iv) *External forces on intermediate stiffeners.* When vertical intermediate stiffeners are subject to bending moments and shears due to the eccentricity of vertical loads, or the action of transverse forces, the moment of inertia of the stiffeners given by i) above shall be increased as shown below:

Bending moment on stiffener due to eccentricity of vertical loading with respect to the vertical axis of the web

$$\text{Increase of } I = \frac{150\ MD^2}{Et}\ \text{cm}^4$$

Lateral loading on stiffener:

$$\text{Increase of } I = \frac{0{\cdot}3\ PD^3}{Et}\ \text{cm}^4$$

where M = the applied bending moment in kNm.

P = the lateral force in kN to be taken by the stiffener and deemed to be applied at the compression flange of the girder.

D = overall depth of girder, in mm

t = thickness of web, in mm.

E = Young's modulus (N/mm^2).

v) *Connections of intermediate stiffeners to web.* Intermediate vertical and horizontal stiffeners not subjected to external loads shall be connected to the web by rivets or welds, in order to withstand a shearing force, in kN/mm run, between each component of the stiffener and the web of not less than

$$\frac{t^2}{8h}$$

where t is the web thickness in mm and h is the outstand of stiffener in mm.

For stiffeners subjected to external loads the shear between the web and stiffeners due to these loads shall be added to the above value.

c) **Stiffeners for tubes.** Where a tubular steel beam rests on an abutment it shall be provided with a shoe adequate to transmit the load to the abutment and to stiffen the tube.

Where a concentrated load is applied to a tubular member consideration shall be given to the local stresses set up and the method of application and stiffening shall be such as to prevent the local stress from being excessive.

29 Filler joists

a) **Bending moment.** The bending moments on slabs of which filler joists form part shall be calculated to satisfy the following:

1) As slabs spanning continuously over supports and subject to those combinations of dead and live load producing the maximum positive and negative moments; or

2) In the case of three or more approximately equal spans of continuous filler joists as described in Subclause *f*), as slabs designed for uniformly distributed loading, satisfying the bending moment values given in Table 15.

NOTE Spans may be considered approximately equal when the longest span does not exceed the shortest span by more than 15 per cent.

Table 15 — Bending moment values for filler joists

Near middle of end span	At support next to end support	At middle of interior spans	At other interior supports
$+\frac{wL^2}{10}$	$-\frac{wL^2}{10}$	$+\frac{wL^2}{12}$	$-\frac{wL^2}{12}$

where w = the dead plus live load per unit length of span.

L = the span centre to centre of supports.

For single spans the slab shall be assumed to be freely supported.

b) **Resistance moment.** The resistance moment of the slab shall be calculated from the section properties of the filler joists only, except that where the filler joists are completely embedded in a solid concrete slab the slab may alternatively be calculated as a composite reinforced concrete section.

c) **Spacing.** The spacing of filler joists centre-to-centre shall not exceed n times the minimum thickness of the structural concrete slabs as given in Table 16, unless the concrete is reinforced to span as a slab, or function as an arch between the filler joists.

Where a slab of concrete is designed to function as an arch between the filler joists, the thrust from the arch action shall be taken up by steel ties or by other means so as not to cause appreciable increase of stress in the joists. The concrete shall have a works strength of not less than 30 N/mm^2 at 28 days, when tested in accordance with BS 1881[5)].

[5)] BS 1881, "Methods of testing concrete."

Table 16 — Spacing of filler joists

Imposed load kN/m^2	Up to 2.5	Above 2.5 and up to 5.0	Above 5.0 and up to 7.5	Above 7.5 and up to 10.0	Above 10.0
Value of n	9	8	7	6	5

d) **Thickness of concrete.** Where the underside of the concrete is arched between the filler joists, the thickness at the crown shall be not less than 50 mm. The thickness of structural concrete over hollow blocks shall nowhere be less than 30 mm when the filler joists do not exceed 500 mm centre-to-centre and nowhere less than 50 mm when the spacing of the filler joists is greater than 500 mm centre-to-centre.

e) **Stresses.** i) *Bending stress.* The flexural stress in filler joists of grade 43 steel, other than those designed as part of a reinforced concrete slab, shall not exceed (165 + 0.6 t) N/mm^2, where t is an allowance the value of which is equal to the thickness in millimetres of the structural concrete cover to the compression flanges of the filler joists; provided that the allowance t is applied only in cases where the filler joists are embedded at least flush with the underside of their bottom flanges in a solid concrete slab throughout and that any cover less than 25 mm and in excess of 75 mm shall be neglected.

In cases where the underside of the slab is flush with the bottom flanges of the filler joists, the allowance t shall not apply in respect of support moments; nevertheless if the top flange is covered the allowance may be made in calculating the resistance to the sagging moment.

ii) *Shear and bearing stress.* The shear and bearing stresses at end supports of grade 43 steel shall be calculated as taken entirely on the filler joists, and the stresses shall not exceed those given for grade 43 steel in Subclause **23** *b*) and Clause **22** respectively.

f) **Joists in continuous fillers.** Continuous fillers shall extend, where possible, over three spans, and any joints shall be made over supports in such a manner as to preserve a considerable measure of continuity. The joints shall be staggered so that full continuity of the fillers occurs in 50 per cent of those crossing any support, such continuity being evenly disposed.

g) **Span-depth ratio.** The span of filler joists, centre-to-centre of supports, shall not exceed 35 times the depth from the underside of the joist to the top of the structural concrete or 12 times this depth in the case of cantilever fillers.

C. Design of compression members

30 Axial stresses in struts

a) **Uncased struts.** The calculated average stresses on the gross sectional area of struts axially loaded shall not exceed p_c in Table 17, in which l/r is the effective length [see Subclause **31** *a*)] divided by the appropriate radius of gyration, except that for material over 40 mm thick for grade 43 steel, 63 mm thick for grade 50 steel and 25 mm thick for grade 55 steel, the values given in Table 17 shall be reduced by 10 per cent.

The formula from which the table has been derived is given in Appendix B.

b) **Cased struts.** Struts of single I section or of two channels back to back in contact or spaced apart not less than 20 mm or more than half their depth and battened or laced in accordance with the requirements of Clauses **35** and **36** may be designed as cased struts when the following conditions are fulfilled:

1) The steel strut is unpainted and solidly encased in ordinary dense concrete, with 10 mm aggregate (unless solidity can be obtained with a larger aggregate) and of a works strength not less than 30 N/mm^2 at 28 days when tested in accordance with BS 1881, "Methods of testing concrete".

2) The minimum width of solid casing is equal to b + 100 mm, where b is the width overall of the steel flange or flanges in millimetres.

3) The surface and edges of the steel strut have a concrete cover of not less than 50 mm.

4) The casing is effectively reinforced with wire complying with BS 4482, "Hard drawn mild steel wire for the reinforcement of concrete". The wire shall be of at least 5 mm diameter and the reinforcement shall be in the form of stirrups or binding at not more than 200 mm pitch, and so arranged as to pass through the centre of the covering to the edges and outer faces of the flanges, and to be supported by and attached to longitudinal spacing bars not fewer than 4 in number. Alternatively, the casing may be reinforced with fabric complying with BS 4483, "Steel fabric for the reinforcement of concrete", or with bars complying with BS 4449, "Hot rolled steel bars for the reinforcement of concrete", provided in either case that the same requirements of diameter, spacing and positioning are met.

Table 17a — Allowable stress p_c on gross section for axial compression

l/r	p_c (N/mm^2) for grade 43 steel									
	0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
0	170	169	169	168	168	167	167	166	166	165
10	165	164	164	163	163	162	162	161	160	160
20	159	159	158	158	157	157	156	156	155	155
30	154	154	153	153	153	152	152	151	151	150
40	150	149	149	148	148	147	146	146	145	144
50	144	143	142	141	140	139	139	138	137	136
60	135	134	133	131	130	129	128	127	126	124
70	123	122	120	119	118	116	115	114	112	111
80	109	108	107	105	104	102	101	100	98	97
90	95	94	93	91	90	89	87	86	85	84
100	82	81	80	79	78	77	75	74	73	72
110	71	70	69	68	67	66	65	64	63	62
120	62	61	60	59	58	57	57	56	55	54
130	54	53	52	51	51	50	49	49	48	47
140	47	46	46	45	45	44	43	43	42	42
150	41	41	40	40	39	39	38	38	38	37
160	37	36	36	35	35	35	34	34	33	33
170	33	32	32	32	31	31	31	30	30	30
180	29	29	29	28	28	28	28	27	27	27
190	26	26	26	26	25	25	25	25	24	24
200	24	24	24	23	23	23	23	22	22	22
210	22	22	21	21	21	21	21	20	20	20
220	20	20	20	19	19	19	19	19	19	18
230	18	18	18	18	18	18	17	17	17	17
240	17	17	17	16	16	16	16	16	16	16
250	16	15	15	15	15	15	15	15	15	15
300	11	11	11	11	11	11	10	10	10	10
350	8	8	8	8	8	8	8	8	8	8

NOTE 1 Intermediate values may be obtained by linear interpolation.
NOTE 2 For material over 40 mm thick refer to subclause **30** *a*).

Table 17b — Allowable stress p_c on gross section for axial compression

l/r	p_0 (N/mm^2) for grade 50 steel									
	0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
0	215	214	214	213	213	212	212	211	211	210
10	210	209	209	208	208	207	207	206	206	203
20	205	204	204	203	203	202	202	201	201	200
30	200	199	199	198	197	197	196	196	195	194
40	193	193	192	191	190	189	188	187	186	185
50	184	183	181	180	179	177	176	174	173	171
60	169	168	166	164	162	160	158	156	154	152
70	150	148	146	144	142	140	138	135	133	131
80	129	127	125	123	121	119	117	115	113	111
90	109	107	106	104	102	100	99	97	95	94
100	92	91	89	88	86	85	84	82	81	80
110	78	77	76	75	74	72	71	70	69	68
120	67	66	65	64	63	62	61	60	60	59
130	58	57	56	55	55	54	53	52	52	51
140	50	50	49	48	48	47	47	46	45	45
150	44	44	43	43	42	42	41	41	40	40
160	39	39	38	38	37	37	36	36	36	35
170	35	34	34	34	33	33	33	32	32	31
180	31	31	30	30	30	30	29	29	29	28
190	28	28	27	27	27	27	26	26	26	26
200	25	25	25	25	24	24	24	24	23	23
210	23	23	23	22	22	22	22	22	21	21
220	21	21	21	20	20	20	20	20	20	19
230	19	19	19	19	19	18	18	18	18	18
240	18	18	17	17	17	17	17	17	17	16
250	16									
300	11									
350	8									

Intermediate values may be obtained by linear interpolation.

NOTE For material over 63 mm thick refer to subclause **30** *a*).

Table 17c — Allowable stress p_c on gross section for axial compression

l/r	p_0 (N/mm^2) for grade 55 steel									
	0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
0	265	264	264	263	262	262	261	260	260	259
10	258	258	257	256	256	255	254	254	253	252
20	252	251	250	250	249	248	248	247	246	246
30	245	244	244	243	242	241	240	239	239	238
40	236	235	234	233	232	230	229	227	226	224
50	222	220	219	217	214	212	210	208	205	203
60	200	197	195	192	189	186	183	180	178	175
70	172	169	166	163	160	157	154	151	148	146
80	143	140	138	135	133	130	128	125	123	121
90	118	116	114	112	110	108	106	104	102	100
100	99	97	95	93	92	90	89	87	86	84
110	83	82	80	79	78	76	75	74	73	72
120	71	69	68	67	66	65	64	63	62	62
130	61	60	59	58	57	56	56	55	54	53
140	53	52	51	50	50	49	49	48	47	47
150	46	45	45	44	44	43	43	42	42	41
160	41	40	40	39	39	38	38	37	37	37
170	36	36	35	35	34	34	34	33	33	33
180	32	32	32	31	31	31	30	30	30	29
190	29	29	28	28	28	28	27	27	27	27
200	26	26	26	25	25	25	25	25	24	24
210	24	24	23	23	23	23	23	22	22	22
220	22	22	21	21	21	21	21	20	20	20
230	20	20	20	19	19	19	19	19	19	18
240	18	18	18	18	18	18	17	17	17	17
250	17									
300	12									
350	9									
Intermediate values may be obtained by linear interpolation.										
NOTE For material over 25 mm thick refer to subclause **30** *a*).										

The radius of gyration r of the strut section about the axis in the plane of its web or webs may be taken as 0·2 (b + 100) mm. The radius of gyration about its other axis shall be taken as that of the uncased section.

In no case shall the axial load on a cased strut exceed twice that which would be permitted on the uncased section, nor shall the slenderness ratio of the uncased section, measured over its full length centre-to-centre of connections, exceed 250.

In computing the allowable axial load on the cased strut the concrete shall be taken as assisting in carrying the load over its rectangular cross section, any cover in excess of 75 mm from the overall dimensions of the steel section of the cased strut being ignored. This cross section of concrete shall be taken as assisting in carrying the load on the basis of a stress equal to the allowable stress in the steel (as given in Table 17) divided by 0·19 times the numerical value of p_{bc} given in Table 2 for the grade of steel concerned.

NOTE This clause does not apply to steel struts of overall sectional dimensions greater than 1 000 mm × 500 mm, the dimension of 1 000 mm being measured parallel to the web, or to box sections.

c) **Angles as struts.** i) For single-angle struts connected to gussets or to a section either by riveting or bolting by not less than two rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially-loaded member provided that the calculated average stress does not exceed the allowable stresses given in Table 17, in which l/r is taken as the greatest of:

1) $0.85L_{vv}/r_{vv}$ but $\geq 0.7L_{vv}/r_{vv} + 15$
2) $1.0L_{aa}/r_{aa}$ but $\geq 0.7L_{aa}/r_{aa} + 30$
3) $0.85L_{bb}/r_{bb}$ but $\geq 0.7L_{bb}/r_{bb} + 30$

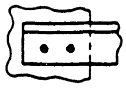
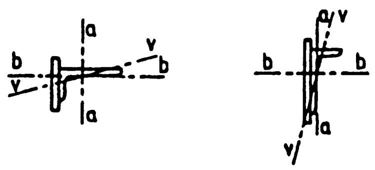
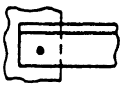
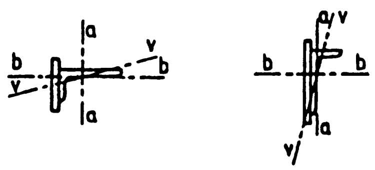
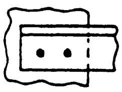
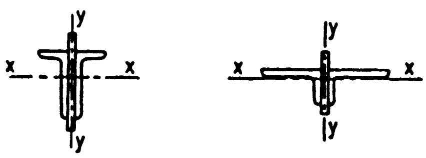
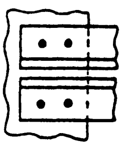
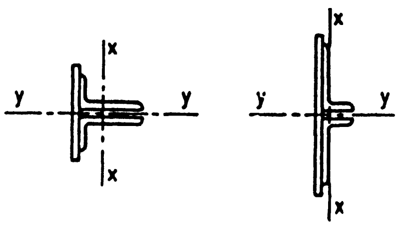
The length of L shall be taken as the distance between the intersections of centroidal axes or the intersections of the setting out lines of the rivets or bolts and r is the radius of gyration about the relevant axis. Axes are defined in Table 18.

Intermediate lateral restraints shall be allowed for in determining the length L for buckling about the relevant axis.

Single angle struts with single-bolted or riveted connections shall be treated similarly, but the ratio of slenderness l/r shall not exceed 180. The calculated stress for such single angle struts shall not exceed 80 per cent of the values given in Table 17 and l/r shall be taken as the greatest of:

1) $1.0L_{vv}/r_{vv}$ but $\geq 0.7L_{vv}/r_{vv} + 15$
2) $1.0L_{aa}/r_{aa}$ but $\geq 0.7L_{aa}/r_{aa} + 30$
3) $1.0L_{bb}/r_{bb}$ but $\geq 0.7L_{bb}/r_{bb} + 30$

Table 18 — Angle Struts

Connection	**Sections and axes**	**Slenderness ratios** (see notes 1 and 2)
		*vv axis:* $0.85L_{vv}/r_{vv}$ but $\geq 0.7L_{vv}/r_{vv} + 15$ *aa axis:* $1.0L_{aa}/r_{aa}$ but $\geq 0.7L_{aa}/r_{aa} + 30$ *bb axis:* $0.85L_{bb}/r_{bb}$ but $\geq 0.7L_{bb}/r_{bb} + 30$
 (See note 3)		*vv axis:* $1.0L_{vv}/r_{vv}$ but $\geq 0.7L_{vv}/r_{vv} + 15$ *aa axis:* $1.0L_{aa}/r_{aa}$ but $\geq 0.7L_{aa}/r_{aa} + 30$ *bb axis:* $1.0L_{bb}/r_{bb}$ but $\geq 0.7L_{bb}/r_{bb} + 30$ (See note 3)
 (See note 4)		*xx axis:* $0.85L_{xx}/r_{xx}$ but $\geq 0.7L_{xx}/r_{xx} + 30$ *yy axis:* $1.0L_{yy}/r_{yy} + 10$
 (Scc notc 4)		*xx axis:* $1.0L_{xx}/r_{xx}$ but $\geq 0.7L_{xx}/r_{xx} + 30$ *yy axis:* $0.85L_{yy}/r_{yy}$ but $\geq 0.7L_{yy}/r_{yy} + 10$

NOTE 1 The length L is taken between the intersections of the centroidal axes or the intersections of the setting out lines of the bolts, irrespective of whether the strut is connected to a gusset or directly to another member.
NOTE 2 Intermediate lateral restraints reduce the value of L for buckling about the relevant axes. For single angle members, L_{vv} is taken between lateral restraints perpendicular to either aa or bb.
NOTE 3 For single angles connected by one bolt, the allowable stress is also reduced to 80 per cent of that for an axially loaded member.
NOTE 4 Double angles are interconnected back-to-back to satisfy Clause **37**.

ii) For double angle discontinuous struts, back-to-back connected to both sides of a gusset or section by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The angles shall be connected together in their length so as to satisfy the requirements of Clause **37**. The calculated average stress shall not exceed the values obtained from Table 17 for ratio of slenderness *l*/*r* taken as the greater of:

1) $0.85L_{xx}/r_{xx}$ but $\geqslant 0.7L_{xx}/r_{xx} + 30$

2) $1.0L_{yy}/r_{yy} + 10$

iii) Double angle discontinuous struts back to back, connected to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, may be regarded as axially-loaded. The angles shall be connected together in their length so as to satisfy the requirements of Clause **37**. The calculated average stress shall not exceed the values obtained from Table 17 for a ratio of slenderness *l*/*r* taken as the greater of:

1) $.0L_{xx}/r_{xx}$ but $\geqslant 0.7L_{xx}/r_{xx} + 30$

2) $0.85L_{yy}/r_{yy} + 10$

iv) The provisions in this clause are not intended to apply to continuous angle struts such as those forming the rafters of trusses, the flanges of trussed girders, or the legs of towers, which shall be designed in accordance with Clause **26** and Table 17.

31 Effective length of compression members

a) **Struts.** For the purpose of calculating *l*/*r* for struts (see Clause **30**) the effective length *l* shall be taken as follows:

1) Effectively held in position and restrained in direction at both ends $l = 0{\cdot}7L$

2) Effectively held in position at both ends and restrained in direction at one end $l = 0{\cdot}85L$

3) Effectively held in position at both ends, but not restrained in direction $l = L$

4) Effectively held in position and restrained in direction at one end, and at the other partially restrained in direction but not held in position $l = 1{\cdot}5L$

5) Effectively held in position and restrained in direction at one end, but not held in position or restrained in direction at the other end $l = 2{\cdot}0L$

where L = Length of strut from centre-to-centre of intersections with supporting members.

b) **Stanchions.** Appendix D (Figure 6 to Figure 20) gives typical cases of how the effective length of a stanchion, as affected by its end and beam connections, shall be assessed for the purpose of this British Standard.

The stanchions shown in the examples for single-storey buildings act as "propped" cantilevers, and the values given in the figures do not necessarily apply to stanchions in which the reactions at the caps of the stanchions due to wind or other horizontal forces are resisted by horizontal girders.

At the caps of stanchions in single-storey buildings, the horizontal deflections due to lateral forces shall not exceed 1/325 of the height of the stanchion except in cases where greater deflections would not impair the strength and efficiency of the structure or lead to damage to finishings.

The use of the effective lengths stated for the examples of stanchions in single-storey buildings is conditional upon:

1) The bases of the stanchions being properly held in position and restrained in direction; and

2) The caps of the stanchions being properly held in position by the provision of diagonal or portal bracing in a vertical plane in at least one longitudinal bay in each line, or by other adequate means.

In cases where the beam connections are eccentric with respect to the axes of the stanchion, the same conditions of restraint shall be deemed to apply, provided the connections are carried across the flange or web of the stanchion as the case may be, and the web of the beam falls within, or is in direct contact with, the stanchion section.

Where practical difficulties prevent this, the effective length shall be estimated to accord with the case appropriate to no restraint in direction.

32 Design details

a) **General.** The thickness of an outstanding leg of any member in compression, unless the leg is stiffened, shall be not less than one-sixteenth of the outstand for grade 43 steel and not less than one-fourteenth of the outstand for grade 50 steel and not less than one-twelfth of the outstand for grade 55 steel.

Unless effectively stiffened, the unsupported width of a plate forming any part of a member primarily in compression, measured between adjacent lines of rivets, bolts or welds connecting the plates to other parts of the section, shall not exceed the following:

90t for grade 43 steel
80t for grade 50 steel
70t for grade 55 steel.

where t is the thickness of a single plate, or the total thickness of two or more plates effectively tacked together (see Clause **51**).

However in calculating the effective area and effective section modulus, the unsupported width of a plate shall be taken as not more than the following:

	grade 43	grade 50	grade 55
welded section	32t	28t	24t
bolted or rivetted	40t	35t	30t

In calculating the radius of gyration and other sectional properties the full area of the plate shall be taken.

b) **Joints.** Where the ends of compression members are faced for bearing over the whole area, they shall be spliced to hold the connected members accurately in place and to resist any tension where bending is present.

Where such members are not faced for complete bearing the joints shall be designed to transmit all the forces to which they are subjected.

Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axis of the members jointed, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stresses shall be provided for.

33 Maximum slenderness ratio of struts

The ratio of the effective length, or of the length centre-to-centre of connections, to the appropriate radius of gyration, shall not exceed the following values:

1. For any member carrying loads resulting from dead weights with or without imposed loads and for single bolted or riveted single angle struts.	180
2. For any member carrying loads resulting from wind forces only, and provided that the deformation of such member does not cause an increase of stress, in any part of the structure, beyond the permissible stress.	250

[See also Subclause **44** *a*)].

34 Eccentricity for stanchions and solid columns

a) For the purpose of determining the stress in a stanchion or column section, the beam reactions or similar loads shall be assumed to be applied 100 mm from the face of the section or at the centre of the bearing, whichever dimension gives the greater eccentricity, and with the exception of the following two cases:

1) In the case of cap connections, the load shall be assumed to be applied at the face of the column shaft or stanchion section, or edge of packing if used, towards the span of the beam.

2) In the case of a roof truss bearing on a cap, no eccentricity need be taken for simple bearings without connections capable of developing an appreciable moment.

b.) In effectively jointed and continuous stanchions the bending moments due to eccentricities of loading at any one floor or horizontal frame level may be taken as being:

1) Ineffective at the floor or frame levels above and below that floor.

2) Divided equally between the stanchion lengths above and below that floor or frame level, provided that the moment of inertia of either stanchion section, divided by its actual length, does not exceed 1·5 times the corresponding value for the other length. In cases exceeding this ratio the bending moment shall be divided in proportion to the moments of inertia of the stanchion sections, divided by their respective actual lengths.

35 Lacing and battening

a) **General.** Struts composed of two main components laced and tied should, where practicable, have a radius of gyration about the axis perpendicular to the plane of the lacing not less than the radius of gyration at right angles to that axis.

As far as practicable the lacing system shall not be varied throughout the length of the strut. The lacing of compression members shall be proportioned to resist a total transverse shear force F_q at any point in the length of the member equal to 2½ per cent of the axial force in the member, which shear force shall be considered as divided equally among all transverse lacing systems in parallel planes.

Except for tie plates as specified in Subclause *h*) below, double intersection lacing systems and single intersection lacing systems mutually opposed in direction on opposite sides of the main components shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the strut unless all forces resulting from deformation of the strut members are calculated and provided for in the lacing and its fastenings.

For members carrying the bending stress calculated from the eccentricity of loading, applied end moments or lateral loading, the lacing shall be proportioned to resist any shear due to the bending, in addition to the above mentioned 2½ per cent.

b) **Determination of section of lacing bars.** The required section of lacing bars for compression members, or for tension members subject to bending, shall be determined by using the appropriate permissible stresses, subject to the requirements in Subclauses *c*) and *d*) below.

For tension members under direct stress only, the lacing bars shall be subject to the requirements of Subclauses *c*), *d*) and *e*) below.

The ratio l/r of the lacing bars for compression members shall not exceed 140.

In riveted construction, the effective length of lacing bars for the determination of the permissible stress shall be taken as the length between the inner end rivets of the bar for single intersection lacing, and as 0·7 times this length for double intersection lacing effectively riveted at intersections.

In welded construction these effective lengths shall be taken as the distance between the inner ends of effective lengths of weld connecting the bars to the members and 0·7 times this length respectively.

c) **Width of lacing bars.** In riveted construction the minimum width of lacing bars shall be:

65 mm for 22 mm diameter rivets

60 mm for 20 mm diameter rivets

50 mm for 16 mm diameter rivets

d) **Thickness of lacing bars.** The thickness of flat lacing bars shall be not less than 1/40 of the length between the inner end rivets or welds for single intersection lacing, and 1/60 of this length for double intersection lacing riveted or welded at the intersections. Rolled sections or tubes of equivalent strength may be used instead of flats.

e) **Angle of inclination.** Lacing bars, whether in double or single intersection systems, shall be inclined at an angle of not less than 40° nor more than 70° to the axis of the member.

f) **Spacing.** The maximum spacing of lacing bars, whether connected by riveting or welding, shall be such that the maximum slenderness ratio l/r of the components of the strut between consecutive connections is not greater than 50, or 0·7 times the most unfavourable slenderness ratio of the member as a whole, whichever is the less, where l is the distance between the centres of the connections of the lacing bars to each component.

Lacing bars shall be so connected that there is no appreciable interruption in the triangulation of the system.

g) **Attachment to main members.** The riveting or welding of lacing bars to the main members shall be sufficient to transmit the load in the bars. Where welded lacing bars overlap the main members, the amount of lap shall be not less than 4 times the thickness of the bar or mean thickness of the flange of the member to which the bars are attached, whichever is the less. The welding shall be provided at least along each side of the bar for the full length of lap.

Where lacing bars are fitted between the main members they shall be connected to each member by fillet welds on each side of the bar or by full penetration butt welds.

h) **Tie plates.** Laced compression members shall be provided with tie plates at the ends of lacing systems and at points where the systems are interrupted.

End tie plates shall have a length of not less than the perpendicular distance between the centroids of the groups of rivets or welds connecting them to the main members, and intermediate tie plates shall have a length of not less than three-quarters of this distance.

The length of a tie plate refers to the dimension measured along the longitudinal axis of the member.

The pitch of rivets in the plate shall not exceed the values given in Subclause **51** *c*).

Where the tie plates are welded on, the welds shall comply generally with the requirements in Clause **36** for batten plates.

Tie plates and their fastenings shall be capable of carrying the forces for which the lacing system is designed, in accordance with the method for calculating battens.

The thickness of tie plates shall be not less than 1/50 of the distance between the innermost connecting lines of rivets or welds, except where they are stiffened on their edges; where such stiffening is provided the plates shall be not less than 8 mm thick.

j) As an alternative to the tie plates described in Subclause *h*) above, a crossbraced panel of the same effective strength may be used.

36 Battened compression members

a) Compression members should preferably have their two main components of the same cross section and symmetrically disposed about their X-X axis. They shall comply with the following:

1) The battens shall be placed opposite each other at each end of the member and at points where the member is stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout.

2) The number of battens shall be such that the member is divided into not less than three bays within its actual length centre-to-centre of connections.

b) In battened compression members in which the ratio of slenderness about the Y-Y axis (axis perpendicular to the battens) is not more than 0·8 times the ratio of slenderness about the X-X axis, the spacing of battens centre-to-centre of end fastenings shall be such that the ratio of slenderness l/r of the lesser main component over that distance shall be not greater than 50 or greater than 0·7 times the ratio of slenderness of the members as a whole, about its X-X axis (axis parallel to the battens).

In battened compression members in which the ratio of slenderness about the Y-Y axis is more than 0·8 times the ratio of slenderness about the X-X axis, the spacing of battens centre-to-centre of end fastenings shall be such that the ratio of slenderness l/r of the lesser main component over that distance shall be not greater than 40 or greater than 0·6 times the ratio of slenderness of the member as a whole about its weaker axis.

c) The battens shall be designed to carry the bending moments and shears arising from a transverse shear force F_q of 2½ per cent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens.

d) Battens shall be of plates, channels or I-sections and shall be riveted or welded to the main components so as to resist simultaneously a longitudinal shear force $F_1 = F_q d/na$ and a moment $M = F_q d/2n$

where d = the longitudinal distance centre-to-centre of battens.

a = the minimum transverse distance between the centroids of the rivet groups or welding,

F_q = the transverse shear force as defined in *c*) above.

n = the number of parallel planes of battens.

e) End battens and those at points where the member is stayed in its length shall have an effective length, longitudinally, of not less than the perpendicular distance between the centroids of the main members, and intermediate battens shall have an effective length of not less than three-quarters of this distance, but in no case shall the effective length of any batten be less than twice the width of one member in the plane of the battens.

f) The effective length of a batten shall be taken as the longitudinal distance between end rivets or end welds.

g) Battened plates shall have a thickness of not less than 1/50 of the minimum distance between the innermost lines of connecting rivet groups or welds except where they are stiffened at their edges. Where channels or I-sections are used as battens with their flanges perpendicular to the main members this requirement does not apply.

h) The length of weld connecting each longitudinal edge of the batten plate to a member shall, in the aggregate, be not less than half the length of the batten plate, and at least one-third of the weld shall be placed at each end of the longitudinal edge. In addition, the welding shall be returned along the ends of the plate for a length equal to at least four times the thickness of the plate.

Where the tie or batten plates are fitted between main members they shall be connected to each member by fillet welds on each side of the plate, equal in length to at least that specified in the preceding paragraph, or by complete penetration butt welds.

j) Battened compression members composed of two angles forming a cruciform cross section shall conform to the above requirements except as follows:

1) The battens shall be in pairs placed one against the other, unless they are welded to form cruciform battens.

2) A transverse shear force of $F_q/\sqrt{2}$ shall be taken as occurring separately about each rectangular axis of the whole member.

3) A longitudinal shear force of $F_1/\sqrt{2}$ and the moment $M/\sqrt{2}$ shall likewise be taken in respect of each batten in each of the two planes, except where the maximum l/r can occur about a rectangular axis, in which case each batten shall be designed to resist a shear of 2½ per cent of the total axial force.

F_q, F_1 and M are as defined in Subclause *d* above, with $n = 1$.

k) Battened compression members not complying with these requirements or those subjected, in the plane of the battens, to eccentricity of loading, applied moments or lateral forces, shall be designed according to the exact theory of elastic stability or empirically from the verification of tests, so that they have a load factor of not less than 1·7 in the actual structure.

37 Compression members composed of two components back-to-back

Compression members composed of two angles, channels or tees, back-to-back in contact or separated by a small distance shall be connected together by riveting, bolting or welding so that the maximum ratio of slenderness l/r of each member between the connections is not greater than 40 or greater than 0·6 times the most unfavourable ratio of slenderness of the strut as a whole, whichever is the less.

In no case shall the ends of the strut be connected together with less than two rivets or bolts or their equivalent in welding, and there shall be not less than two additional connections spaced equidistant in the length of the strut. Where the members are separated back-to-back the rivets or bolts through these connections shall pass through solid washers or packings, and where the legs of the connected angles or tables of the connected tees are 125 mm wide or over, or where webs of channels are 150 mm wide or over not less than two rivets or bolts shall be used in each connection, one on the line of each gauge mark.

Where these connections are made by welding, solid packings shall be used to effect the jointing unless the members axe sufficiently close together to permit welding, and the members shall be connected by welding along both pairs of edges of the main components.

The rivets, bolts or welds in these connections shall be sufficient to carry the shear forces and moments (if any) specified for battened struts, and in no case shall the rivets or bolts be less than 16 mm diameter for members up to and including 10 mm thick; 20 mm diameter for members up to and including 16 mm thick; and 22 mm diameter for members over 16 mm thick.

Compression members connected by such riveting, bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the washer-riveted, bolted or welded surfaces.

Where the components are in contact back-to-back, the spacing of the rivets, bolts or intermittent welds shall not exceed the maximum spacing for compression members as given in Subclauses **51** *c*) i) and **54** *c*).

38 Stanchion and column bases

a) **Gusseted bases.** For stanchions with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with the bearing area of the shaft shall, where all the parts are fabricated flush for bearing, be sufficient to take the loads, bending moments and reactions to the base plate without exceeding the specified stresses.

Where the ends of the stanchion shaft and the gusset plates are not faced for complete bearing, the fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

b) **Slab bases.** Stanchions with slab bases need not be provided with gussets, but fastenings shall be provided sufficient to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. When the slab alone will distribute the load uniformly, the minimum thickness of a rectangular slab shall be:

$$t = \sqrt{\left\{\frac{3w}{p_{bct}}\left(A^2 - \frac{B^2}{4}\right)\right\}}$$

where t = the slab thickness in mm.
A = the greater projection of the plate beyond the stanchion in mm.
B = the lesser projection of the plate beyond the stanchion in mm.
w = the pressure or loading on the underside of the base in N/mm².
p_{bct} = the permissible bending stress in the steel (185 N/mm² — see Table 2.)

When the slab will not distribute the load uniformly or when the slab is not rectangular, special calculations shall be made to show that the stresses are within the special limits.

For solid round steel columns in cases where loading on the cap or under the base is uniformly distributed over the whole area including the column shaft, the minimum thickness, in mm, of a square cap or base shall be:

$$t = 10 \times \sqrt{\left(\frac{90W}{16p_{bct}} \cdot \frac{D}{D-d}\right)}$$

where t = the thickness of the plate in mm.

W = the total axial loading in kN.

D = the length of the side of cap or base in mm.

d = the diameter of the reduced end, if any, of the column in mm.

p_{bct} = the permissible bending stress in the steel (185 N/mm$^{2)}$).

When the load on the cap or under the base is not uniformly distributed or where the end of the column shaft is not machined with the cap or base, or where the cap or base is not square in plan, calculations shall be made based on the allowable stress of 185 N/mm^2.

The cap or base plate shall be not less than 1·5 (d + 75) mm in length or diameter.

The area of the shoulder (the annular bearing area) shall be sufficient to limit the stress in bearing, for the whole of the load communicated to the slab, to the maximum values given in Clause **22** and resistance to any bending communicated to the shaft by the slab shall be taken as assisted by bearing pressures developed against the reduced end of the shaft in conjunction with the shoulder.

Bases for bearing upon concrete or masonry need not be machined on the underside provided the reduced end of the shaft terminates short of the surface of the slab, and in all cases the area of the reduced end shall be neglected in calculating the bearing pressure from the base.

In cases where the cap or base is welded direct to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required above for fastenings to slab bases.

39 Base plates and bearing plates

The base plates and grillages of stanchions and the bearings and spreaders of beams and girders shall be of adequate strength, stiffness and area to spread the load upon the concrete, masonry, other foundation, or other support, without exceeding the permissible stress on such foundation under any combination of load and bending moments.

40 Grillage foundations

Where grillage beams are enveloped in a solid block of dense concrete as specified in condition 2 in Clause **21**, the permissible working stresses specified in this British Standard for uncased beams may be increased by 33⅓ per cent provided that:

a) The beams are spaced apart so that the distance between the edges of adjacent flanges is not less than 75 mm,

b) The thickness of the concrete cover on the top of the upper flanges, at the ends, and at the outer edges of the sides of the outermost beams is not less than 100 mm;

c) The concrete is properly compacted solid around all beams.

These increased stresses shall not apply to hollow compound girders.

D. Design of tension members

41 Axial stresses in tension

The direct stress in axial tension P_t on the net area of section shall not exceed the value given in Table 19.

Table 19 — Allowable stress p_t in axial tension

Form	Grade	Thickness	P_t
		mm	N/mm^2
Sections, bars, plates, wide flats and hot rolled hollow sections	43	≤ 40	170
		over 40 but ≤ 100	155
	50	≤ 63	215
		over 63 but ≤ 100	200
	55	≤ 25	265

42 Tensile stresses for angles, tees and channels

a) **Eccentric connections.** When eccentricity of loading occurs in connections of angles and tees in tension, the net areas to be used in computing the mean tensile stress shall be as given by the following rules:

1) *Single angles connected through one leg, channel sections connected through the web and T-sections connected only through the flange.* To the net sectional area of the connected leg, add the sectional area of the unconnected leg multiplied by:

$$\frac{3a_1}{3a_1 + a_2}$$

where a_1 = the sectional area of the connected leg.
a_2 = the sectional area of the unconnected leg.

Where lug angles are used, the net sectional area of the whole of the angle member shall be taken.

2) *A pair of angles, channels or T-sections, connected together along their length,* when attached to the same side of a gusset for the equivalent by only one leg of each component:

i) in contact or separated, by a distance not exceeding the aggregate thickness of the connected parts, with solid packing pieces.

ii) connected by bolts or welding as specified in Subclauses **51** *e*) or **54** *g*) so that the maximum ratio of slenderness of each member between connections is not greater than 80.

To the net sectional area of the connected part, add the sectional area of the unconnected part multiplied by:

$$\frac{5a_1}{5a_1 + a_2}$$

where a_1 = the net sectional area of the connected part.
a_2 = the sectional area of the unconnected part.

Where the components are widely spaced, this rule does not apply, and the member shall be specially designed.

b) **Double angles, tees or channels placed back-to-back** and connected to each side of a gusset or to each side of part of a rolled section. For computing the mean tensile stress the net sectional area of the pair shall be taken, provided the members are connected together along their length as specified in Subclause **51** *e*) or **54** *g*).

NOTE The area of the leg of an angle shall be taken as the product of the thickness by the length from the outer corner minus half the thickness, and the area of the leg of a tee as the product of the thickness by the depth minus the thickness of the table.

For *lacing* and *battening* of tension members see Subclause **35** b).

E. Constructional details

43 Braced frames and trusses

a) i) Members of braced frames and trusses shall, where practicable, be disposed symmetrically about the resultant line of force, and the connections shall, where practicable, be arranged so that their centroid lies on the resultant of the forces they are intended to resist [see Subclause **48** *c*)].

ii) In the case of bolted, riveted or welded trusses and braced frames, the strut members act under complex conditions and the effective length l shall be taken between 0·7 and 1·0 times the distance between centres of intersections, depending on the degree of end restraint [but see also Subclause **30** *c*].

b) Where braced frames or trusses are supported by walls or piers, they shall be secured thereto where necessary for anchorage.

c) Tension members which are subject to reversal of stress due to temperature changes or vibration shall be designed to have lateral rigidity.

44 Roof trusses

a) For any member normally acting as a tie in a roof truss but subject to reversal of stress resulting from the action of wind, the ratio of the effective length to the least radius of gyration shall not exceed 350.

b) The windward roof trusses in multiple-bay buildings shall be designed to resist the appropriate wind forces estimated as set out in CP 3:Chapter V, “Wind loads”, and the component members of the sheltered trusses, if the trusses are of the same height, span, rise and spacing as the windward truss, shall be of the same sections as those of the windward trusses.

Where, in multiple-bay buildings, a sheltered truss is either of different height, span, rise or spacing from the windward truss the component members of the sheltered truss shall be proportioned as if the sheltered truss were a windward truss. Where, however, the wind loads produce greater forces or reversal of forces in any members, such members shall be proportioned to resist these greater forces or reversed forces.

45 Purlins

All purlins shall be designed in accordance with the requirements for uncased beams (Subclause **19** *a*) and Clause **25**).

NOTE The provisions of Clause **26** regarding lateral stability and of Clause **15** limiting the deflection of beams do not apply to purlins.

Alternatively, in the case of roof slopes not exceeding 30° pitch, purlins may be designed in accordance with the following empirical rules, which are based on a minimum imposed loading of 0·75 kN/m^2. The deflections obtained under these rules may be found to exceed those permitted by Clause **15** but should in any case be limited to suit the characteristics of the roof covering.

In these rules L is the centre-to-centre distance in millimetres of the steel principals or other support of the purlins, and W is the total distributed load, in kN, on the purlin arising from dead load and snow, but excluding wind, both assumed as acting normal to the roof.

Angle purlins of grade 43 *steel for roof slopes not exceeding* 30° *pitch.* The leg or the depth of the purlin taken approximately in the plane of action of the maximum load or maximum component of the load shall be not less than $L/45$; the other leg or width of the purlin shall be not less than $L/60$, and the numerical value of the section modulus of the purlin in centimetre units shall be not less than $WL/1{\cdot}8 \times 10^{-3}$.

Tubular purlins of grade 43 *steel for roof slopes up to and including* 30° *pitch.* The diameter of the purlins shall be not less than $L/64$ and the numerical value of the section modulus of the purlins in centimetre units shall be not less than $WL/2 \times 10^{-3}$.

NOTE Purlins designed in accordance with these rules shall be deemed to be capable of carrying the forces resulting from the requirements of Subclause **26** *e*) ii).

46 Side and end sheeting rails

Side and end sheeting rails shall be designed for wind pressures and vertical loads, if any; and the requirements of Clauses **15** and **26** as regards limiting deflection and lateral stability of beams do not apply.

47 Steel castings

The use of steel castings shall be limited to bearings, junctions and other similar parts and the working stresses shall not exceed the working stresses given in this British Standard for Grade 43 steel sections.

Steel castings shall conform to BS 592[6)], "Carbon steel castings for general engineering purposes", Grade A, 430 N/mm^2 with a minimum yield stress of 215 N/mm^2.

48 Connections

a) **Rivets, close tolerance bolts, high strength friction grip bolts[7)], black bolts and welding.** As much of the work of fabrication as is reasonably practicable shall be completed in the shops where the steelwork is fabricated.

Where a connection is subject to impact or vibration or to reversal of stress (unless such reversal is due solely to wind), or where for some special reason — such as continuity in rigid framing or precision in alignment of machinery — slipping of bolts is not permissible, then rivets, close tolerance bolts, high strength friction grip bolts or welding shall be used. In all other cases bolts in clearance holes may be used provided that due allowance is made for any slippage.

b) **Composite connections.** In any connection which takes a force directly communicated to it and which is made with more than one type of fastening, only rivets and close tolerance bolts shall be considered as acting together to share the load. In all other connections sufficient of one type of fastening shall be provided to communicate the entire force.

c) **Members meeting at a joint.** For triangulated frames, designed on the assumption of pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axes meeting at a point; and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections.

However, where eccentricity of members or of connections is present, the members and the connections shall provide adequate resistance to the induced bending moments.

[6)] Included in BS 3100, "Steel castings for general engineering purposes".

[7)] BS 4395, "High strength friction grip bolts and associated nuts and washers for structural engineering. Metric series", Part 1, "General grade", Part 2, "Higher grade bolts and nuts and general grade washers".
BS 4604, "The use of high strength friction grip bolts in structural steelwork. Metric series", Part 1, "General grade", Part 2, "Higher grade (parallel shank)".

Where the design is based on non-intersecting members at a joint, all stresses arising from the eccentricity of the members shall be calculated and the stresses kept within the limits specified in the appropriate clauses of this British Standard.

d) **Packings.**

i) *Rivets or bolts through packings.* The number of rivets or bolts carrying shear through packing shall be increased above the number required by normal calculations by 1¼ per cent for each 1 mm total thickness of packing, except that, for packings having a thickness of 6 mm or less, no increase need be made.

For double shear connections packed on both sides the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing.

The additional rivets or bolts may be placed in an extension of the packing.

ii) *Packing in welded construction.* Where a packing is used between two parts, the packing and the welds connecting it to each part shall be capable of transmitting the load between the parts, except where the packing is too thin to carry the load or permit the provision of adequate welds, when the load shall be transmitted through the welds alone, the welds being increased in size by an amount equal to the thickness of the packing.

49 Lug angles

Lug angles connecting a channel-shaped member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

In the case of angle members the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength not less than 20 per cent in excess of the force in the outstanding leg of the angle, and the attachment of the lug angle to the angle member shall be capable of developing 40 per cent in excess of that force.

In the case of channel members and the like, the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10 per cent in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20 per cent in excess of that force.

In no case shall fewer than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

The effective connection of the lug angle shall, as far as possible, terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.

Where lug angles are used to connect an angle member the whole area of the member shall be taken as effective, notwithstanding the requirements of Clause **42**.

F. Riveting and bolting

50 Allowable stresses in rivets and bolts

a) **Calculation of stresses.** In calculating shear and bearing stresses the effective diameter of a rivet shall be taken as the hole diameter, and that of a bolt as its nominal diameter. In calculating the axial tensile stress in a rivet the gross area [see Subclause **17** *b*)] shall be used, and in calculating that in a bolt or a screwed tension rod the net area [see Subclause **17** *c*)] shall be used.

b) **Stresses in rivets and bolts.** The calculated stress in a mild steel rivet or in a bolt of strength grade 4.6 shall not exceed the value given in Table 20.

The allowable calculated stress in a high tensile steel rivet shall be that given in Table 20 multiplied by the ratio of the tensile strength of the rivet material to 400 N/mm^2.

The allowable calculated stress in a bolt (other than a high strength friction grip bolt) of higher grade than 4.6 shall be that given in Table 20 multiplied by the ratio of its yield stress (or its stress at the permanent set limit of 0.2 %) or 0.7 times its tensile strength, whichever is the lesser, to 2.35 N/mm^2.

Table 20 — Allowable stresses in rivets and bolts (N/mm^2)

Description of fasteners	Axial tension	Shear	Bearing
Power-driven rivets	100	100	300
Hand-driven rivets	80	80	250
Close tolerance and turned bolts	120	100	300
Bolts in clearance holes	120	80	250

c) **Bearing stresses on connected parts.** The calculated bearing stress of a rivet or bolt on the parts connected by it shall not exceed the value given in Table 20A.

Where the end distance of a rivet or bolt (i.e. the edge distance in the direction in which it bears) is less than a limit of twice the effective diameter of the rivet or bolt, the allowable bearing stress of that rivet or bolt on the connected part shall be reduced in the ratio of the actual end distance to that limit.

Table 20A — Allowable bearing stresses on connected parts (N/mm^2)

Descripton of fasteners	Material of connected part		
	Grade 43	Grade 50	Grade 55
Power-driven rivets Close tolerance and turned bolts	300	420	480
Hand-driven rivets Bolts in clearance holes	250	350	400

d) **Combined shear and tension.** Rivets and bolts subject to both shear and axial tension shall be so proportioned that the calculated shear and axial stresses f_s and f_t, calculated in accordance with Subclause **50** *a*), do not exceed the respective allowable stresses p_s and p_t and that the quantity $f_s/p_s + f_t/p_t$ does not exceed 1.4.

e) **High strength friction grip bolts.** The preceding subclauses *a*) to *d*) do not apply to high strength friction grip bolts, which shall only be used in conformity with BS 4604.

51 Rivets and riveting

a) **Rivets.** Rivets shall conform to the requirements of BS 4620[8)] for dimensions.

b) **Minimum pitch.** The distance between centres of rivets shall be not less than 2½ times the nominal diameter of the rivet.

c) **Maximum pitch.** i) The distance between centres of any two adjacent rivets (including tacking rivets) connecting together elements of compression or tension members shall not exceed 32*t* or 300 mm where *t* is the thickness of the thinner outside plate.

ii) The distance between centres of two adjacent rivets, in a line lying in the direction of stress, shall not exceed 16*t* or 200 mm in tension members, and 12*t* or 200 mm in compression members. In the case of compression members in which forces are transferred through butting faces this distance shall not exceed 4½ times the diameter of the rivets for a distance from the abutting faces equal to 1½ times the width of the member.

iii) The distance between centres of any two consecutive rivets in a line adjacent and parallel to an edge of an outside plate shall not exceed 100 mm + 4*t*, or 200 mm in compression or tension members.

iv) When rivets are staggered at equal intervals and the gauge does not exceed 75 mm the distances specified in ii) and iii) above, between centres of rivets, may be increased by 50 per cent.

d) **Edge distance.** i) The minimum distance from the centre of any hole to the edge of a plate shall be in accordance with Table 21.

ii) Where two or more parts are connected together a line of rivets or bolts shall be provided at a distance of not more than 40 mm + 4t from the nearest edge, where t is the thickness in millimetres of the thinner outside plate. In the case of work not exposed to weather, this may be increased to 12t.

Table 21 — Edge distance of holes

Diameter of hole	Distance to sheared or hand flame cut edge	Distance to rolled, machine flame cut, sawn or planed edge
mm	mm	mm
39	68	62
36	62	56
33	56	50
30	50	44
26	42	36
24	38	32
22	34	30
20	30	28
18	28	26
16	26	24
14	24	22
12 or less	22	20

e) **Tacking rivets.** Where tacking rivets are necessary to satisfy the requirements of *d*) ii) above, such tacking rivets, not subject to calculated stress, shall have a pitch in line not exceeding 32 times the thickness of the outside plate or 300 mm whichever is the less. Where the plates are exposed to the weather, the pitch in line shall not exceed 16 times the thickness of the outside plate or 200 mm whichever is the less. In both cases, the lines of rivets shall not be a greater distance apart than these pitches.

8) BS 4620. Rivets for general engineering purposes. Metric series

The foregoing requirements shall apply to struts and compression members generally, subject to the stipulations in this British Standard affecting the design and construction of struts.

In tension members composed of two flats, angles, channels or tees in contact back-to-back or separated back-to-back by a distance not exceeding the aggregate thickness of the connected parts, tacking rivets, with solid distance pieces where the parts are separated, shall be provided at a pitch in line not exceeding 1 000 mm.

f) **Countersunk heads.** For countersunk heads one-half of the depth of the countersinking shall be neglected in calculating the length of the rivet in bearing. For rivets in tension with countersunk heads the tensile value shall be reduced by 33⅓ per cent. No reduction need be made in shear.

g) **Long grip rivets.** Where the grip of rivets carrying calculated loads exceeds 6 times the diameter of the holes, the number of rivets required by normal calculation shall be increased by not less than ⅝ per cent per additional 1 mm of grip; but the grip shall not exceed 8 times the diameter of the holes.

52 Bolts and bolting

a) **Pitches, edge distances and tacking bolts.** The requirements for bolts shall be the same as specified for rivets in Subclauses **51** *b*), *c*), *d*) and *e*).

b) **Black bolts (black all over).** The dimensions shall conform to those given for black bolts in BS 4190, "ISO metric black hexagon bolts, screws and nuts."

c) **Close tolerance bolts.** The dimensions shall conform to those given for bolts "faced under the head and turned on shank" in BS 4190, or to those given for bolts in BS 3692 provided that threads are kept clear of connected parts in accordance with Clause **62**.

d) **Turned barrel bolts.** The nominal diameter of the barrel shall be in multiples of 2 mm and shall be at least 2 mm larger in diameter than the screwed portion.

e) **Locking of nuts.** Wherever there is a risk of nuts becoming loose due to vibration or alternation of stresses, they shall be securely locked.

G. Welds and welding

53 Allowable stresses in welds

a) **General.** When electrodes of designations E43 ... R or E51 ... B(H) to BS 639 are used for the welding of grade 43 or grade 50 steel or designation E51 ... B(H) to BS 639 is used for the welding of grade 55 steel and the yield stress of an all-weld tensile test specimen is not less than 430 N/mm^2 when tested in accordance with Appendix D of BS 639[9], the following shall apply.

i) *Butt welds.* Butt welds shall be treated as parent metal with a thickness equal to the throat thickness (or a reduced throat thickness as specified in Clause **54** for certain butt welds) and the stresses shall not exceed those allowed for the parent metal.

ii) *Fillet welds.* The allowable stress in fillet welds, based on a thickness equal to the throat thickness shall be:

1) 125 N/mm^2 for grade 43 plates;

2) 160 N/mm^2 for grade 50 steel;

3) 195 N/mm^2 for grade 55 steel;

iii) When electrodes appropriate to a lower grade of steel are used for welding together parts of material of a higher grade of steel, the allowable stresses for the lower grade of steel shall apply.

iv) When a weld is subject to a combination of stresses, the stresses shall be combined as required in Subclause **14** *c*) and *d*), the value of the equivalent stress f_c being not greater than that permitted for the parent metal.

b) **Steel tubes.**

i) A weld connecting two tubes end to end shall be a butt weld.

ii) A weld connecting the end of one tube (branch tube) to the surface of another (main tube), with the axis of the tubes intersecting at an angle of not less than 30°, shall be either:

[9] BS 639. "Covered electrodes for the manual metal-arc welding of carbon and carbon manganese steels".

A a butt weld throughout;

or B a fillet weld throughout;

or C a fillet weld in one part and a butt weld in another with a continuous change from the one form to the other in the intervening portions.

Type A may be used whatever the ratio of the diameters of the tubes joined, provided complete penetration is secured either by the use of backing material, or by depositing a sealing run of metal on the back of the joint or by some special method of welding. When Type A is not employed, Type B shall be used where the diameter of the branch tube is less than one-third of the diameter of the main tube, and Type C shall be used where the diameter of the branch tube is equal to or greater than one-third of the diameter of the main tube.

The stress in a butt weld shall be calculated on an area equal to the throat thickness multiplied by the effective length of the weld, measured at the centre of its thickness.

The stress in a fillet weld or a fillet-butt weld (Type C) shall be calculated on an area equal to the throat thickness at the crotch multiplied by the length of the weld (Figure 3). (For a method of calculating the length of the weld, see Appendix C.)

In a fillet weld or in a fillet-butt weld the permissible stress shall not exceed the shear stress permissible in the parent metal.

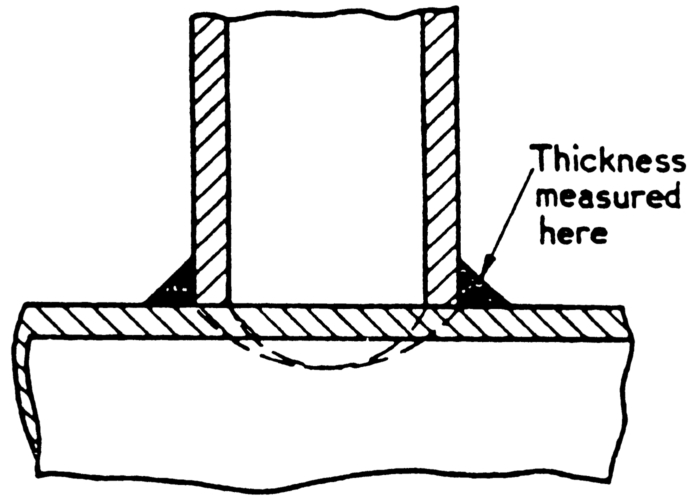
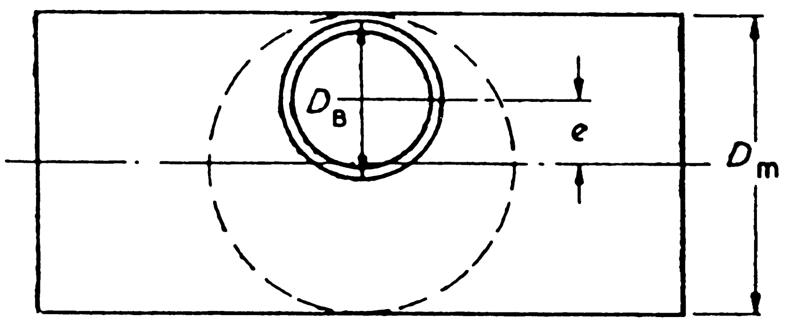
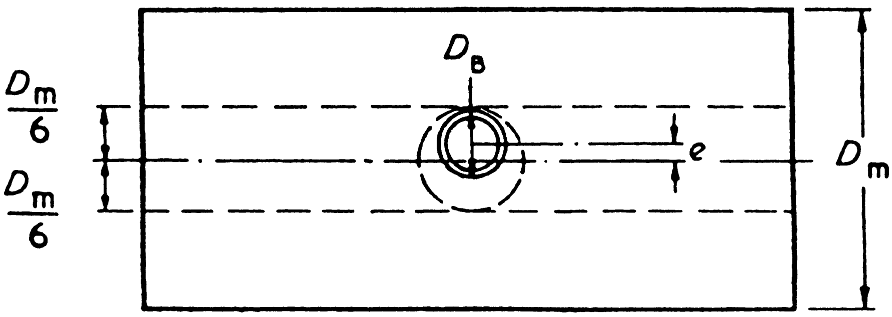


Figure 3 — Illustration of crotch of joint



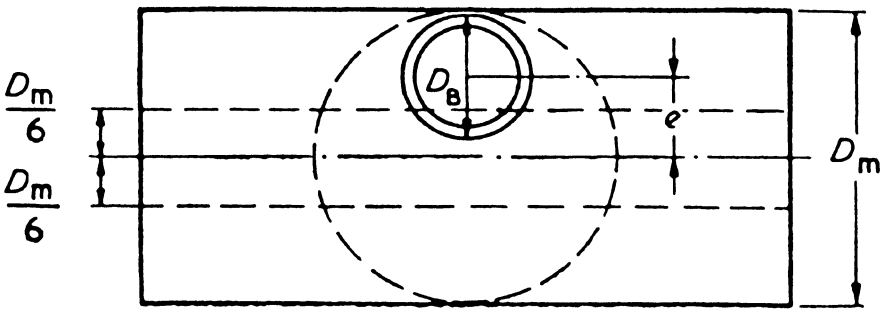
$$e_{max} = ½(D_m - D_B)$$

Figure 4a — Type a butt weld



$$e_{max} = \tfrac{1}{2}\left(\frac{D_m}{3} - D_B\right)$$

Figure 4b — Type b fillet weld



$$e_{max} = \tfrac{1}{2}(D_m - D_B) \quad e_{min} = \tfrac{1}{2}\left(\frac{D_m}{3} - D_B\right)$$

Figure 4c — Type c butt-fillet weld

iii) A weld connecting the end of one tube to the surface of another with the axes of the tubes intersecting at an angle of less than 30° shall be permitted only if adequate efficiency of the junction has been demonstrated.

iv) A weld connecting the end of one tube to the surface of another, where the axes of the two tubes do not intersect, shall be subject to the provisions of Subclauses *b*) ii) and iii) above provided that no part of the curve of intersection of the eccentric tube with the main tube lies outside the curve of intersection of the corresponding largest permissible non-eccentric tube with the main tube (Figure 4a, Figure 4b and Figure 4c).

v) Where the end of the branch tube is flattened to an elliptical or other appropriate shape, Subclauses *b*) ii), iii) and iv) above shall apply, and for the application of Subclauses ii) and iv) the diameter of the flattened portion of the tube shall be measured in a plane perpendicular to the axis of' the main tube.

54 Design of welds

The design of welds shall be in accordance with BS 5135 "Metal-arc welding of carbon and carbon manganese steels" subject to the following conditions, and where fatigue conditions apply, in accordance with the note to clause **7**.

a) **Intermittent butt welds.** Intermittent butt welds shall only be used to resist shear, and the effective length of any such weld shall be not less than 4 times, and the longitudinal space between the effective lengths of welds not more than 12 times, the thickness of the thinner part joined.

b) **Throat thickness of incomplete penetration butt welds** (BS 1856, Clause **13**). Unsealed single V, U, J, or bevel butt welds, and other butt welds which are welded from one side only and are not full penetration welds, shall have a throat thickness of at least seven-eighths of the thickness of the thinner part joined. Evidence shall be produced to show that this throat thickness has been achieved. For the purpose of stress calculation and to allow for the effects of the eccentricity of the weld metal relative to the parts joined, a nominal throat thickness, not exceeding five-eighths of the thickness of the thinner part joined, shall be taken.

c) **Intermittent fillet welds.** The distance along an edge of a part between effective lengths of consecutive intermittent fillet welds, whether the welds are in line or staggered on alternate sides of the edge, shall not exceed 16 times the thickness of the thinner part when in compression nor 24 times the thickness of the thinner part when in tension, and shall in no case exceed 300 mm. This requirement shall not be taken into account in complying with the requirements of *g*) below and Clause **37**.

Intermittent fillet welds shall not be used where they would result in the formation of rust pockets.

d) **Lap joints.** In lap joints, the minimum amount of lap shall be 4 times the thickness of the thinner part connected. Single fillet welds shall be used only where the lapped parts are sufficiently restrained to prevent opening of the joint.

e) **End returns.** Wherever practicable, fillet welds terminating at the ends or sides of parts or members shall be returned continuously around the corners for a distance of not less than twice the size of the weld.

f) **Side fillets.** If side fillets alone are, used in end connections, the length of each side fillet should be not less than the distance between the edges, and the side fillet may be either at the edges of the member or in slots or holes.

g) **Intermittent welding of tension members.** In tension members composed of two flats, angles, channels or tees in contact back-to-back, or separated back-to-back, intermittent welds not subject to calculated stress shall be provided to connect the two parts together along both pairs of edges, with solid distance pieces where necessary where the connected parts are not in contact.
The pitch of the effective lengths of welds, centre-to-centre, shall not exceed 1 000 mm in line whether they are opposite or staggered in respect of the two pairs of edges.

Intermittent welds in tension members shall not be used where they would result in the formation of rust pockets.

Chapter 5. Fabrication and erection

55 Inspection

The purchaser and his authorized representatives shall have access at all reasonable times to all places where the work is being carried out, and shall be provided, by the contractor, with all the necessary facilities for inspection during construction.

H. Work off site

56 Straightness

All material, before and after fabrication, shall be straight unless required to be of curvilinear form, and shall be free from twists.

57 Clearances

Care shall be taken to ensure that the clearances specified are worked to. The erection clearance for cleated ends of members connecting steel to steel shall not be greater than 2 mm at each end. The erection clearance at ends of beams without web cleats shall be not more than 3 mm at each end, but where, for practical reasons, this clearance has to be increased, the seatings shall be suitably designed.

Where black bolts are used the holes may be made not more than 2 mm greater than the diameter of the bolts, for bolts up to 24 mm diameter and not more than 3 mm greater than the diameter of the bolts, for bolts over 24 mm diameter, unless otherwise specified by the Engineer.

58 Cutting

Cutting may be by shearing, cropping, sawing or machine flame cutting. Hand flame cutting may be permitted, subject to the approval of the Engineer.

Sheared or cropped edges shall, if necessary, be dressed to a neat workman-like finish and shall be free from distortion where parts are to be in metal-to-metal contact.

59 Holing

a) **Drilling.** Holes through more than one thickness of material in pre-assembled elements shall, where possible, be drilled after the members are assembled and tightly clamped or bolted together. Where this is possible and when so specified by the Engineer the parts shall be separated after drilling and any burrs removed.

b) **Punching and reaming.** Punching may be permitted before assembly provided that the holes are punched at least 2 mm less in diameter than the required size and are reamed to the full diameter.

c) **Full size punching.** Punching may be permitted full size, but only under the following conditions:

i) Where the punching operation shall not, in the opinion of the Engineer, unduly distort the material.

ii) The holes shall be free of burrs which would prevent solid seating of the parts when tightened.

iii) Holes shall not be punched through material of a thickness greater than the diameter of the hole.

iv) The maximum thickness of material shall be 12 mm for steel subgrade A, 16 mm for steel subgrade B and 20 mm for steel subgrade C for steel graded in accordance with BS 4360.

v) In spliced connections the holes in the mating surfaces shall be punched from the same direction.

Punching full size shall not be permitted under the following conditions:

1) In non-slip connections made with HSFG bolts.
2) At locations where plastic hinges or yield lines are assumed in the design analysis.
3) Where repetition of loading makes fatigue critical to the member design.

d) **Holes for rivets and black bolts.** All matching holes for rivets and black bolts shall register with each other so that a gauge 2 mm less in diameter than the diameter of hole will pass freely through the assembled members in a direction at right angles to such members. Finished holes shall not be more than 2 mm in diameter larger than the diameter of the rivet or black bolt passing through them, for rivet or bolt diameters up to 24 mm, and not more than 3 mm greater than the diameter of the rivet or black bolt for rivet or bolt diameters over 24 mm, unless otherwise specified by the Engineer. Holes for rivets or bolts shall not be formed by a gas cutting process.

e) **Holes for close tolerance and barrel bolts.** Holes for close tolerance and barrel bolts shall be drilled to a diameter equal to the nominal diameter of the shank or barrel subject to a tolerance of + 0.15 mm and – 0 mm. Preferably, parts to be connected with close tolerance or barrel bolts shall be firmly held together by tacking bolts or clamps and the holes drilled through all the thicknesses at one operation and subsequently reamed to size. All holes not drilled through all thicknesses at one operation shall be drilled to a smaller size and reamed out after assembly. Where this is not practicable, the parts shall be drilled and reamed separately through hard bushed steel jigs.

60 Assembly

The component parts shall be assembled in such a manner that they are neither twisted nor otherwise damaged, and shall be so prepared that the specified cambers, if any, are provided.

All tubular members shall be sealed so as to prevent the access of moisture to the inside of the members (see also Clause **62**).

61 Riveting

Rivets shall be heated uniformly throughout their length, without burning or excessive scaling, and shall be of sufficient length to provide a head of standard dimensions. They shall, when driven, completely fill the holes and, if countersunk, the countersinking shall be fully filled by the rivet, any proudness of the countersunk head being dressed off flush, if required.

Riveted members shall have all parts firmly drawn and held together before and during riveting, and special care shall be taken in this respect for all single-riveted connections. For multiple riveted connections, a service bolt shall be provided in every third or fourth hole.

Wherever practicable, machine riveting shall be carried out by using machines of the steady pressure type.

All loose, burned or otherwise defective rivets shall be cut out and replaced before the structure is loaded, and special care shall be taken to inspect all single-riveted connections.

Special care shall be taken in heating and driving long rivets.

62 Bolting

Where necessary, washers shall be tapered or otherwise suitably shaped to give the heads and nuts of bolts a satisfactory bearing.

The threaded portion of each bolt shall project through the nut at least one thread.

A close tolerance or turned bolt shall be of sufficient length to avoid any threaded portion being within the thickness of the connected parts that is required to develop the bearing load on the bolt, and shall be provided with a washer or washers, under the nut, of sufficient thickness to ensure that at least one full thread (in addition to the thread runout) remains clear between the nut and the unthreaded shank.

Where a tubular member is drilled to take bolts or studs, provision shall be made to prevent the access of moisture to the interior of the tube. For example, a transverse sleeve can be inserted where a bolt passes through a tube, or grommets can be used under heads and nuts.

63 Welding

Welding shall be in accordance with BS 5135 “Metal-arc welding of carbon and carbon manganese steels”.

64 Qualification and testing of welders

For welding of any particular type of joint, welders shall give evidence acceptable to the purchaser of having satisfactorily completed appropriate tests as described in Chapter 6. The purchaser may require the tests to be carried out in the presence of his representative.

65 Flattened ends of tubes

For welded, riveted or bolted connections, the ends of tubes may be flattened or otherwise formed provided the methods adopted are such as not to injure or deface the material. The change of section shall be gradual.

66 Machining of butts, caps and bases

Stanchion splices and butt joints of compression members dependent on contact for the transmission of compressive stresses, shall be accurately prepared to butt so that the permitted stress in bearing is not exceeded nor eccentricity of loading created which would induce secondary bending in the members. Stanchion caps and bases shall be prepared in a similar manner to the above, and, where this is obtained by machining, care shall be taken that any attached gussets, connecting angles or channels are fixed with such accuracy that they are not reduced in thickness by more than 2 mm.

67 Slab bases and caps

Slab bases and slab caps, except when cut from material with true surfaces, shall be accurately machined over the bearing surfaces and shall be in effective contact with the end of the stanchion. A bearing face which is to be grouted direct to a foundation need not be machined if such face is true and parallel to the upper face.

To facilitate grouting, holes shall be provided where necessary in stanchion bases for the escape of air.

68 Solid round steel columns

Solid round steel columns with shouldered ends shall be provided with slab caps and bases machined to fit the shoulders, and shall be tightly shrunk on or welded in position.

The tolerance between the reduced end of the shaft and the hole, in the case of slabs welded in position, shall not exceed 0·250 mm.

Where slabs are welded in position the reduced end of the shaft shall be kept just sufficiently short to accommodate a fillet-weld around the hole without weld-metal being proud of the slab.

Alternatively, the caps and bases may be welded direct to the column without boring or shouldering.

All bearing surfaces of slabs intended for metal-to-metal contact shall be machined perpendicular to the shaft.

69 Marking

Each piece of steelwork shall be distinctly marked before delivery, in accordance with a marking diagram, and shall bear such other marks as will facilitate erection.

70 Painting

All surfaces which are to be painted, oiled or otherwise treated shall be dry and thoroughly cleaned to remove all loose scale and loose rust.

Shop contact surfaces need not be painted unless so specified. If so specified, they shall be brought together while the paint is still wet.

Surfaces not in contact, but inaccessible after shop assembly, shall receive the full specified protective treatment before assembly. This does not apply to the interior of sealed hollow sections.

In the case of surfaces to be welded the steel shall not be painted or metal coated within a suitable distance of any edges to be welded if the paint specified or the metal coating would be harmful to welders or impair the quality of the welds.

Welds and adjacent parent metal shall not be painted prior to de-slagging inspection and approval.

Parts to be encased in concrete shall not be painted or oiled.

J. Work on site

71 Plant and equipment

The suitability and capacity of all plant and equipment used for erection shall be to the satisfaction of the Engineer.

72 Storing and handling

All structural steel at the site shall be stored and handled so that members are not subjected to excessive stresses and damage. Attention is drawn to the requirements in Item 4 of Subclause **3** *a*) iii).

73 Setting out

The positioning and levelling of all steelwork, the plumbing of stanchions and the placing of every part of the structure with accuracy shall be in accordance with the approved drawings and to the satisfaction of the Engineer.

74 Security during erection

During erection the work shall be securely bolted or otherwise fastened, and if necessary temporarily braced, so as to make adequate provision for all erection stresses and conditions, including those due to erection equipment and its operation. Neither riveting, permanent bolting nor welding shall be done until proper alignment has been obtained.

75 Painting after erection

Before painting of steel which is delivered unpainted is commenced, all surfaces to be painted shall be dry and thoroughly cleaned from all loose scale and rust.

The specified protective treatment shall be completed after erection. All rivet and bolt heads and site welds after de-slagging shall be cleaned. Damaged or deteriorated paint surfaces shall first be made good with the same type of paint as the shop coat. Where specified, surfaces which will be in contact after site assembly shall receive a coat of paint (in addition to any shop priming) and shall be brought together while the paint is still wet.

Where the steel has received a metal coating in the shop this coating shall be completed on site so as to be continuous over any welds and site rivets or bolts, but subject to the approval of the Engineer protection may be completed by painting on site. Bolts which have been galvanized or similarly treated are exempted from this requirement.

Surfaces which will be inaccessible after site assembly shall receive the full specified protective treatment before assembly.

Site painting should not be done in frosty or foggy weather, or when humidity is such as to cause condensation on the surfaces to be painted.

76 Bedding of stanchion bases, bedding and encasing of grillage beams and bearings of beams and girders

a) **Bedding of stanchion bases and bearings of beams and girders on stone, brick or concrete (plain or reinforced).** Bedding shall be carried out with Portland cement grout or mortar, or fine concrete.

For multi-storey buildings this operation shall not be carried out until a sufficient number of bottom lengths of stanchions have been properly lined, levelled and plumbed and sufficient floor beams are in position.

Whatever method is employed the operation shall not be carried out until the steelwork has been finally levelled and plumbed, the stanchion bases being supported meanwhile by steel wedges; and, immediately before grouting, the space under the steel shall be thoroughly cleaned.

The following methods are permissible:

i) Using neat Portland cement grout to a thickness not exceeding 25 mm: the grout shall be mixed as thickly as possible consistent with fluidity and shall be poured under a suitable head so that the space is completely filled.

ii) Using fluid Portland cement mortar to a thickness of between 25 mm and 50 mm the mortar shall be not leaner than 1 : 2 cement to sand and shall be mixed as thickly as possible consistent with fluidity. It shall be poured under a suitable head and tamped until the space has been completely filled.

iii) Using Portland cement mortar mixed as dry as possible and having a thickness of not less than 50 mm the mortar shall be not leaner than 1 : 2 cement to fine aggregate, and shall be consolidated by thoroughly ramming with a suitable blunt rammer against properly fixed supports, until the space has been completely filled.

b) **Bedding of grillage beams on concrete (plain or reinforced).** The space under grillage beams and the space, where permitted, between two layers of grillage beams shall be filled with grout or mortar as specified for stanchion bases.

Bedding with Portland cement or mortar between stanchion base plates and the top tier of grillage beams shall not be allowed unless the base is specially designed to limit the stresses to those permissible on the mortar.

Where possible, grouting of grillages should not take place until the first lengths of stanchions have been plumbed, aligned and levelled.

c) **Encasing of steelwork in foundations and filling between grillage beams.** Grillage beams and all steel in foundations shall be solidly encased in dense concrete as specified in condition 2 in Clause **21**, with a minimum cover of 100 mm.

Chapter 6. Tests for use in the Approval of Welders

77 Tests

For the welding of structural steelwork, welding procedures and welders shall be tested in accordance with the series of welding standards BS 4870, BS 4871 and BS 4872.

Changes of welding procedure requiring requalification of welders

Extent of qualification of welders

Clauses **78** and **79** deleted July 1975.

Appendix A. Loading Tests

The methods of testing structures of unconventional design referred to in Subclause **9** *c*) of this standard shall be as follows:

Acceptance tests.

The structure or structural member under consideration shall be loaded with the dead load for as long a time as possible before testing and the tests shall be conducted as follows:

i) *Stiffness test.* In this test the structure or member shall be subjected, in addition to the dead load, to a test load equal to 1½ times the imposed load, and this loading shall be maintained for 24 hours. The maximum deflection attained during this test should not be excessive. If, after removal of the test load, the member or structure does not show a recovery of at least 80 per cent of the maximum strain or deflection shown during the 24 hours under load, the test shall be repeated. The structure shall be considered to have sufficient stiffness, provided that the recovery after this second test is not less than 90 per cent of the maximum increase in strain or deflection shown during the second test. The deflection of a beam under its design loading shall comply with Clause **15**.

ii) *Strength test.* The structure shall be subjected, in addition to the dead load, to a test load equal to the sum of the dead load and twice the imposed load, and this load shall be maintained for 24 hours.

In the case of wind load, a load corresponding to twice the wind load shall be applied and maintained for 24 hours, either with or without the vertical test load, according to which condition is the more severe in the member under consideration or the structure as a whole. Complete tests under both conditions may be necessary to verify the strength of the structure. The structure shall be deemed to have adequate strength if during the test no part completely fails, and if on removal of the test load the structure shows a recovery of at least 20 per cent of the maximum deflection or strain shown during the 24 hours under load.

Where several structures are to be built to the same design and it is considered unnecessary to test all of them, one structure, as a prototype, shall be fully tested, as described in i) and ii), by a qualified engineer, but in addition, during the first application of the test load, particular note shall be taken of the strain or deflection when the test load, at 1½ times the imposed load, has been maintained for 24 hours. This information is required as a basis of comparison in any check tests carried out on samples of the structures.

When a structure of the same type is selected for a check test it shall be subjected, in addition to dead load, to an imposed test load, equal to 1½ times the specified live load, in a manner and to an extent prescribed by the engineer carrying out the test. This load shall be maintained for 24 hours, during which time the maximum deflection should be noted. The check test shall be considered to be satisfactory, provided that the maximum strain or deflection noted in the check test does not exceed by more than 20 per cent the maximum strain or deflection shown at similar load in the test on the prototype.

An actual structure which has satisfied test ii), "Strength test", and is subsequently to be erected for use, shall be considered satisfactory for occupancy after it has been strengthened by replacing any distorted members, and has subsequently satisfied test i), "Stiffness test".

NOTE **Method of testing.** The manner in which the loading is to be applied and the positions at which deflections or strains are to be measured can only be decided with reference to the particular structure to be tested; but as a general guide the following are suggested:

Beams and girders. The deflection should be measured at mid span; and if it is expected that considerable strain or settlement may occur at the supports, the deflection or settlement at the supports should also be recorded.

Cantilevers. The deflections at the end of the cantilever should be measured; and, under the conditions visualized above for the beams, the deflections at the support.

Stanchions. The lateral deflections at the mid-height of the stanchion and at the head of the stanchion should be measured relative to the joint next below and to the base.

Loading

i) *Vertical loading.* Uniformly distributed loading on beams may be represented by two loads, each half the total of the uniformly distributed load, applied at the quarter points.

ii) *Horizontal loading.* Where the effect of horizontal forces on the structure as a whole has to be tested, the action of the cladding in applying the load to the frame should be taken into account. The horizontal forces would in general be represented by a limited number of point loads.

iii) *Dead load equivalent.* It has been assumed that the dead load is in place, but when it is convenient to do the test before the dead load is applied, an equivalent can be used, placed as described in i). The deflection due to the dead load or its equivalent should not be included in the test measurements.

Appendix B Formula from which Table 17 has been derived

The average stress on the gross sectional area of a strut or other compression member in steel with a specified minimum yield stress shall not exceed the value of p_c obtained by the formula:

$$K_2 p_c = \frac{Y_s + (\eta + 1)C_0}{2} - \sqrt{\left\{\left(\frac{Y_s + (\eta + 1)C_0}{2}\right)^2 - Y_s C_0\right\}}$$

where p_c = the permissible average stress, N/mm².

K_2 = load factor or coefficient, taken as 1·7 for the purposes of this standard.

Y_s = minimum yield stress, N/mm².

C_0 = Euler critical stress $= \frac{\pi^2 E}{(l/r)^2} = \frac{\pi^2 210\,000}{(l/r)^2}$ N/mm².

η = $0.3\,(l/100r)^2$.

l/r = slenderness ratio = effective length/radius of gyration.

NOTE For values of l/r less than 30 the value of p_c shall not exceed that obtained from linear interpolation between the value of p_c for l/r = 30 as found above, and a value of p_c for l/r=0 of 155 N/mm² for grade 43 steel sections and 170 N/mm² for grade 43 plates[10) and hot rolled hollow sections or 215 N/mm² for grade 50 steel or 265 N/mm² for grade 55 steel.

Appendix C Determination of the length of the curve of intersection of a tube with another tube or with a flat plate

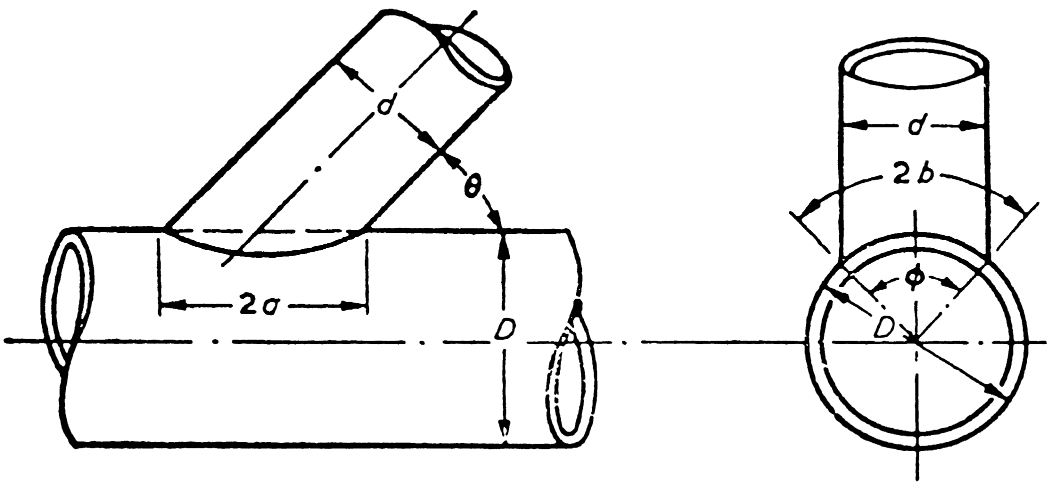


Figure 5 — Determination of length of intersection

The length of the curve of intersection may be taken as:

$$P = a + b + 3\sqrt{a^2 + b^2}$$

where $a = \frac{d}{2}\operatorname{cosec}\theta$

$$b = \frac{d}{3}\cdot\frac{3 - (d/D)^2}{2 - (d/D)^2}$$ ...for intersection with a tube[a]

$$= \frac{d}{2}$$ ...for intersection with a flat plate.

10) The term "plates" denotes plates, wide flats and universal wide flats.

θ = angle between branch and main.
d = outside diameter of branch.
D = outside diameter of main.

[a] Alternatively $b = \frac{D}{4}\phi$

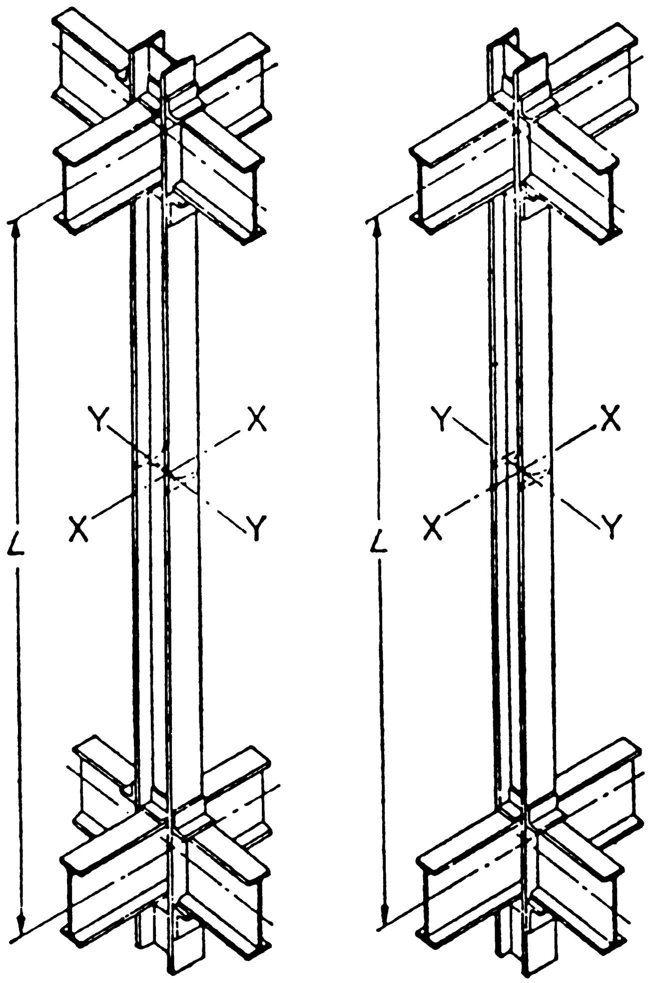
where ϕ is measured in radians and $\sin \frac{\phi}{2} = \frac{d}{D}$

Appendix D [see Subclause 31 *b*)] Effective length of stanchions

Stanchions effectively held in position and restrained in direction at both ends in respect of the weaker axis.

Effective length of stanchions = $0{\cdot}7L$

Slenderness ratio for design = $\frac{0{\cdot}7L}{r_y}$



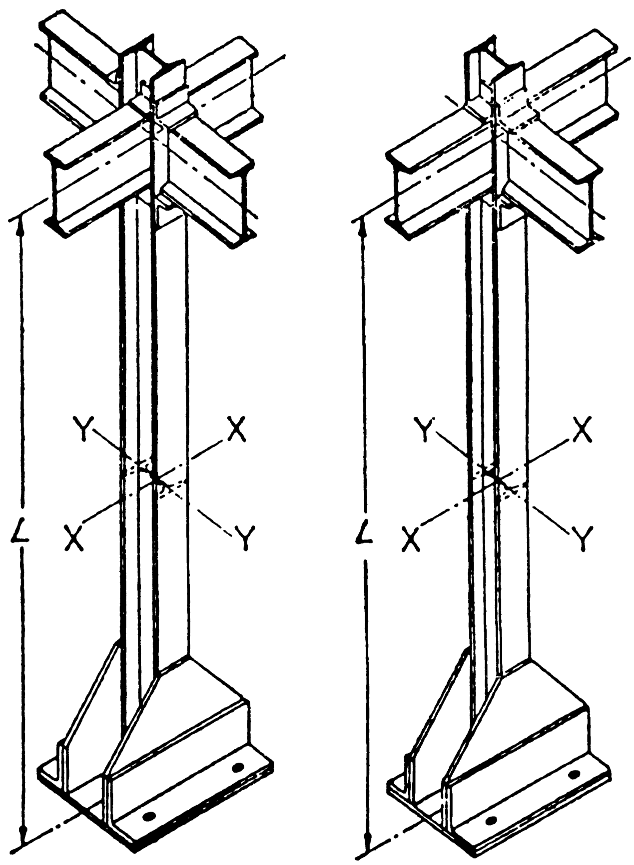
NOTES. Beams are all load-carrying.
Beams forming pairs to be of approximately equal size and span and carrying approximately equal loads.
Beams framing into the web of the stanchions to have moment connections.
All beams to be securely held at their remote ends.

Figure 6 — Effective length of stanchions
Continuous intermediate lengths or top lengths of stanchions

Stanchions effectively held in position and restrained in direction at both ends in respect of the weaker axis.

Effective length of stanchions = 0·7L

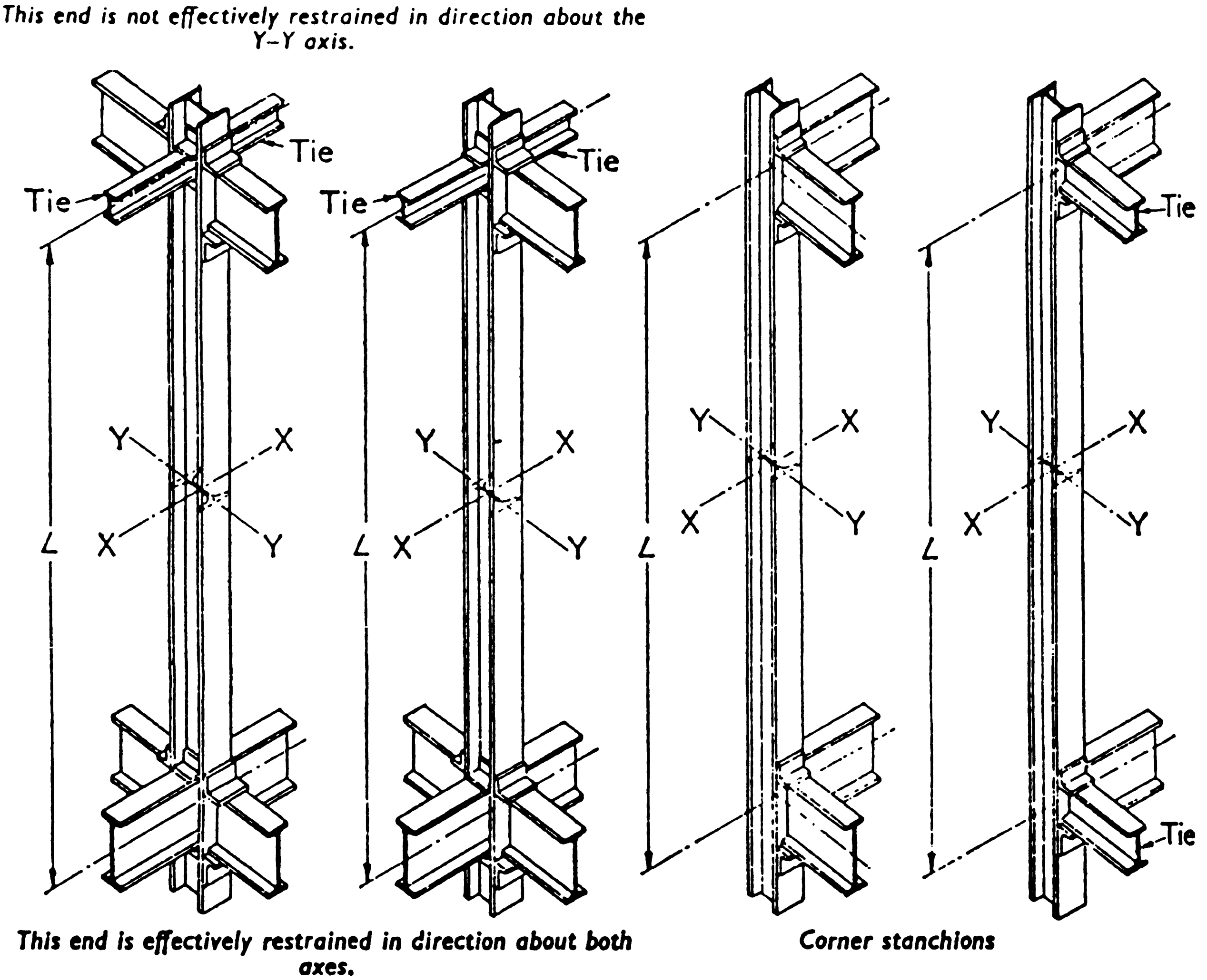
Slenderness ratio for design = $\dfrac{0{\cdot}7L}{r_y}$



Substantial base

NOTES. Beams are all load-carrying.
Beams forming pairs to be of approximately equal size and span and carrying approximately equal loads.
Beams framing into the web of the stanchion to have moment connections.
All beams to be securely held at their remote ends.
The foundation shall be capable of affording restraint commensurate with that of the base.

Figure 7 — Effective length of stanchions
Bottom length of stanchions



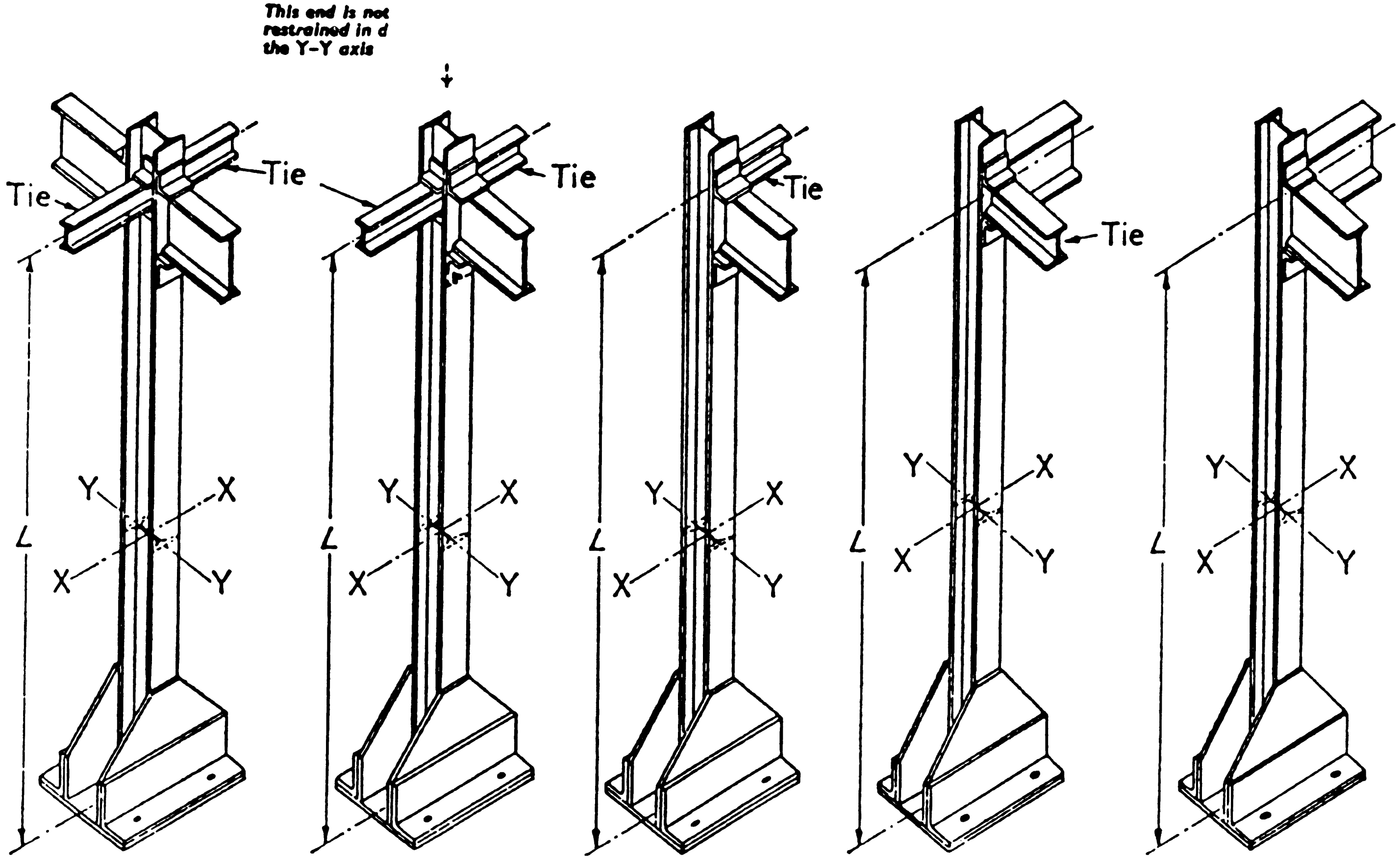
Stanchions effectively held in position at both ends and restrained in direction at one end.

Effective length of stanchions = 0·85*L*

$$\text{Slenderness ratio for design} = \frac{0{\cdot}85L}{r_y}$$

NOTES. Beams are load-carrying except tie beams.
Beams forming pairs to be of approximately equal size and span and carrying approximately equal loads.
All beams shall be securely held at their remote ends.

Figure 8 — Effective length of stanchions
Continuous intermediate lengths or top lengths of stanchions



Substantial base giving restraint in direction about both axes *Corner stanchions*

Stanchions effectively held in position at both ends and restrained in direction at one end.

Effective length of stanchions = 0·85*L* Slenderness ratio for design $= \dfrac{0{\cdot}85L}{r_y}$

NOTES. Beams are load-carrying except tie beams.
Beams forming pairs to be of approximately equal size and span and carrying approximately equal loads.
All beams shall be securely held at their remote ends.
The foundation shall be capable of affording restraint commensurate with that of the base.

Figure 9 — Effective length of stanchions
Bottom length of stanchions

Stanchions effectively held at both ends in position but not restrained in direction (except at the base when this is substantial).

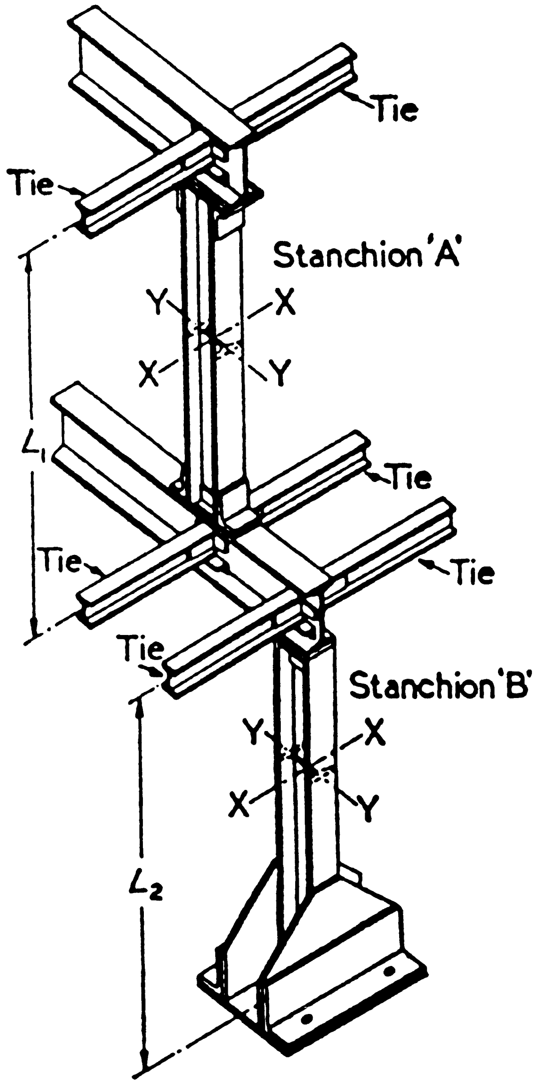
Effective length for stanchion 'A' $= 1{\cdot}0L_1$

Effective length for stanchion 'B' with a small riveted base or slab base $= 1{\cdot}0L_2$

Effective length for stanchion 'B' with a substantial riveted base $= 0{\cdot}85L_2$

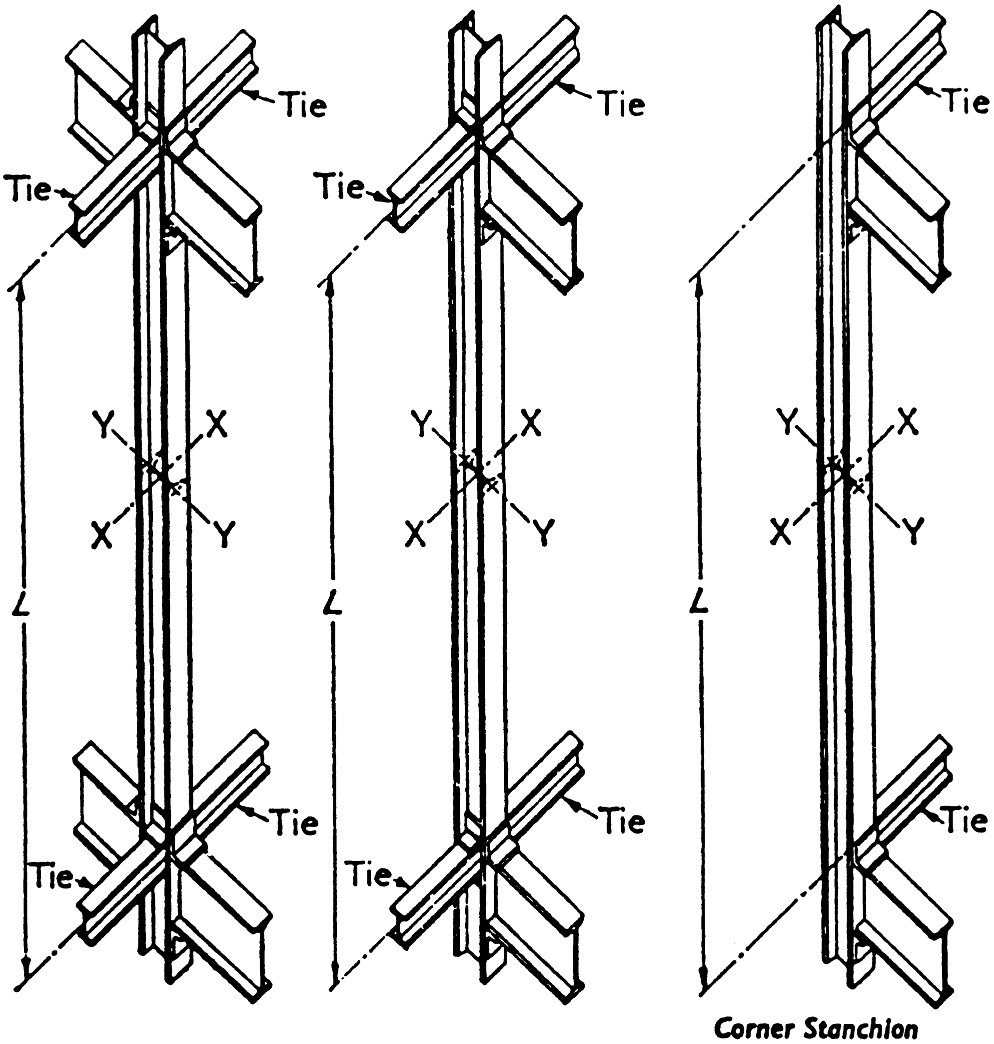
Slenderness ratio for the design of stanchion 'A' $= \dfrac{1{\cdot}0L_1}{r_{1y}}$

Slenderness ratio for the design of stanchion 'B' $= \dfrac{1{\cdot}0L_2}{r_{2y}}$ or $\dfrac{0{\cdot}85L_2}{r_{2y}}$ (depending upon the type of base).



NOTES. All the beams and ties shall be securely held at their remote ends.
Tie beams may have no moment connections.

Figure 10 — Effective length of stanchions Discontinuous stanchions



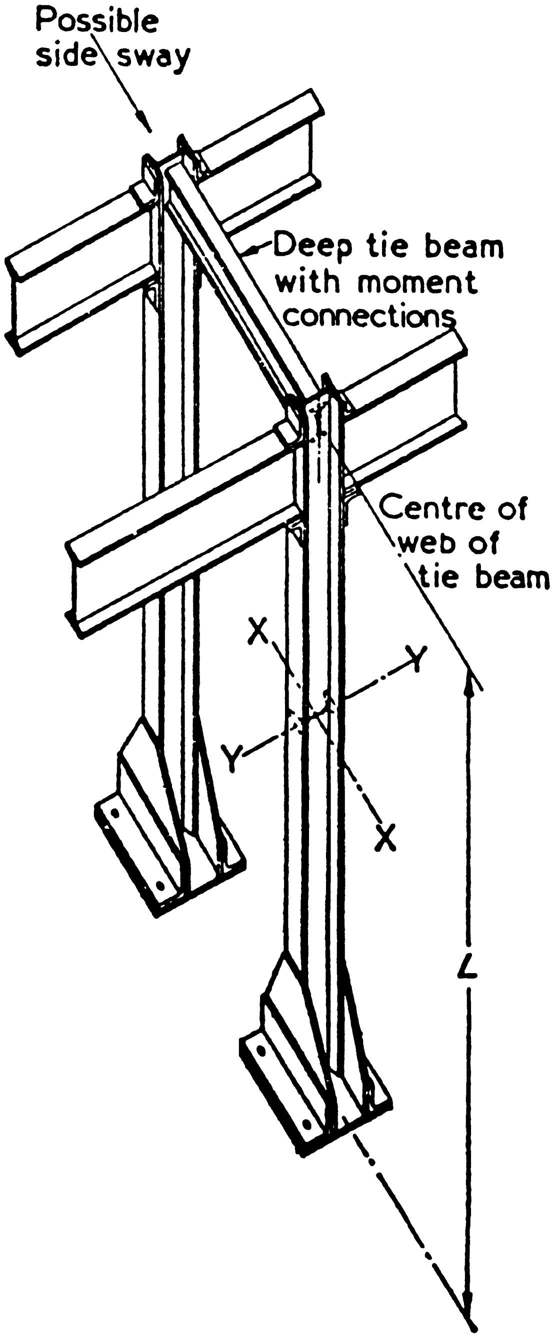
Stanchions effectively held at both ends in position but not restrained in direction (about the weaker axis).

Effective length of stanchions = 1·0*L*

Slenderness ratio for design $= \dfrac{1{\cdot}0L}{r_y}$

NOTES. All the beams shall be securely held at their remote ends.
Tie beams may have no moment connections.

Figure 11 — Effective length of stanchions Continuous intermediate length



Stanchions effectively held at one end in position and direction and at the other end partially restrained in direction but not held in position.

Effective length of stanchions = 1·5L

Slenderness ratio for design = $\frac{1 \cdot 5L}{r_y}$

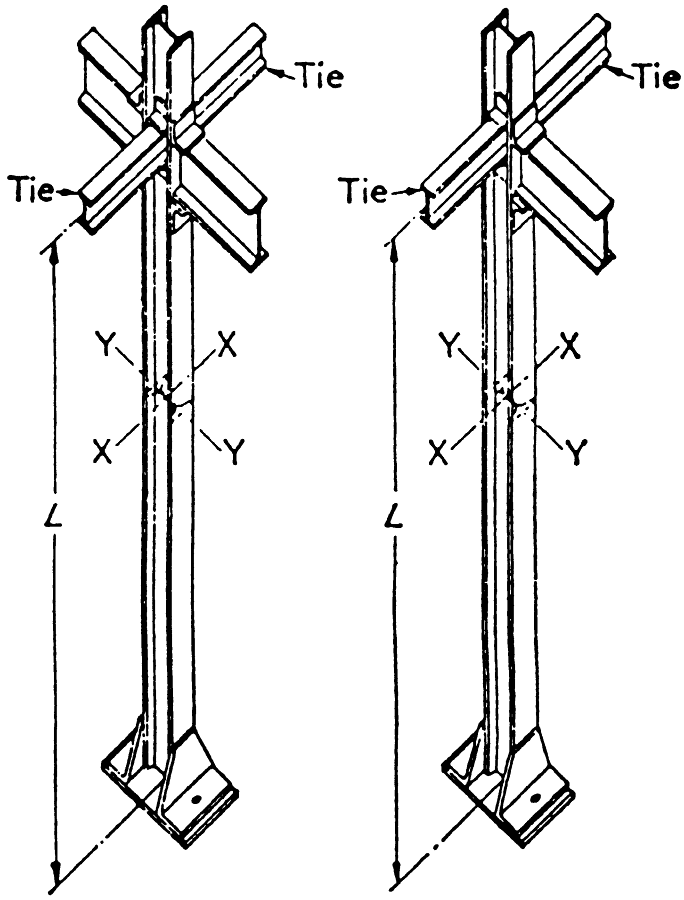
NOTES. All beams shall be securely held at their remote ends.
The foundation shall be capable of affording restraint commensurate with that of the base.

Figure 12 — Effective length of stanchions
Bottom length (single storey)

Stanchions effectively held at both ends in position but not restrained in direction.

Effective length of stanchions = 1·0L

Slenderness ratio for design $= \dfrac{1 \cdot 0L}{r_y}$



Small base, (or slab base)

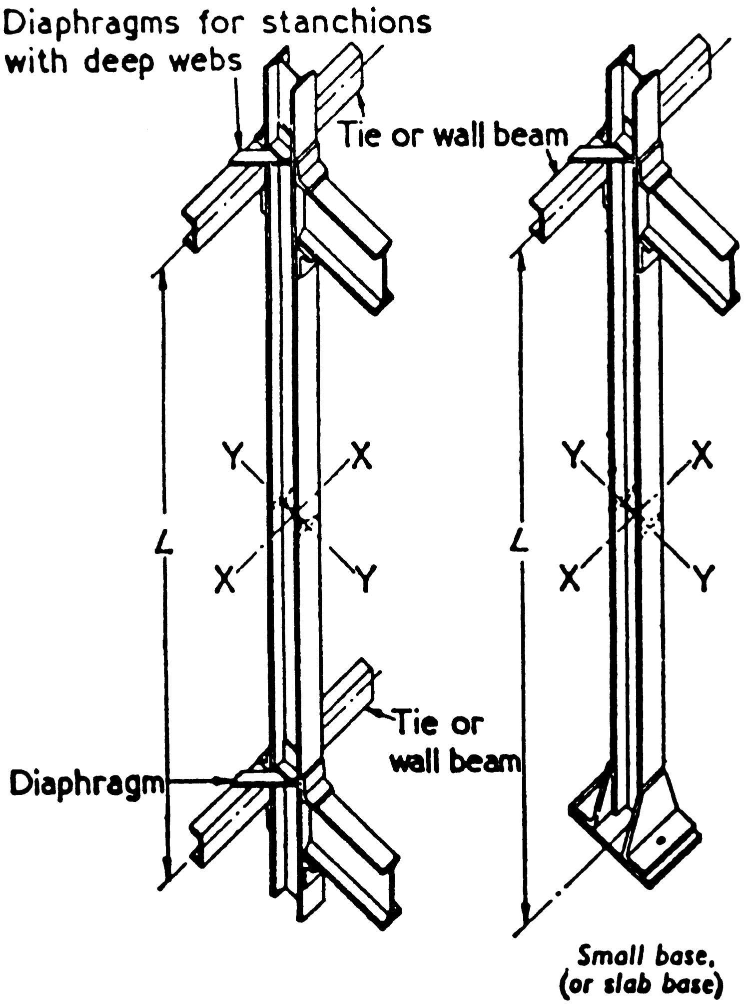
NOTES. All the beams and ties shall be securely held at their remote ends.
Tie beam connections have no appreciable moment restraint.

Figure 13 — Effective length of stanchions
Bottom lengths

Stanchions effectively held at both ends in position but not restrained in direction.

Effective length of stanchions = 1·0L

Slenderness ratio for design $= \dfrac{1 \cdot 0L}{r_y}$



NOTES. All the beams and ties shall be securely held at their remote ends.
Tie beam connections have no appreciable moment restraint.

Figure 14 — Effective length of stanchions
Stanchions with tie beams attached to one flange

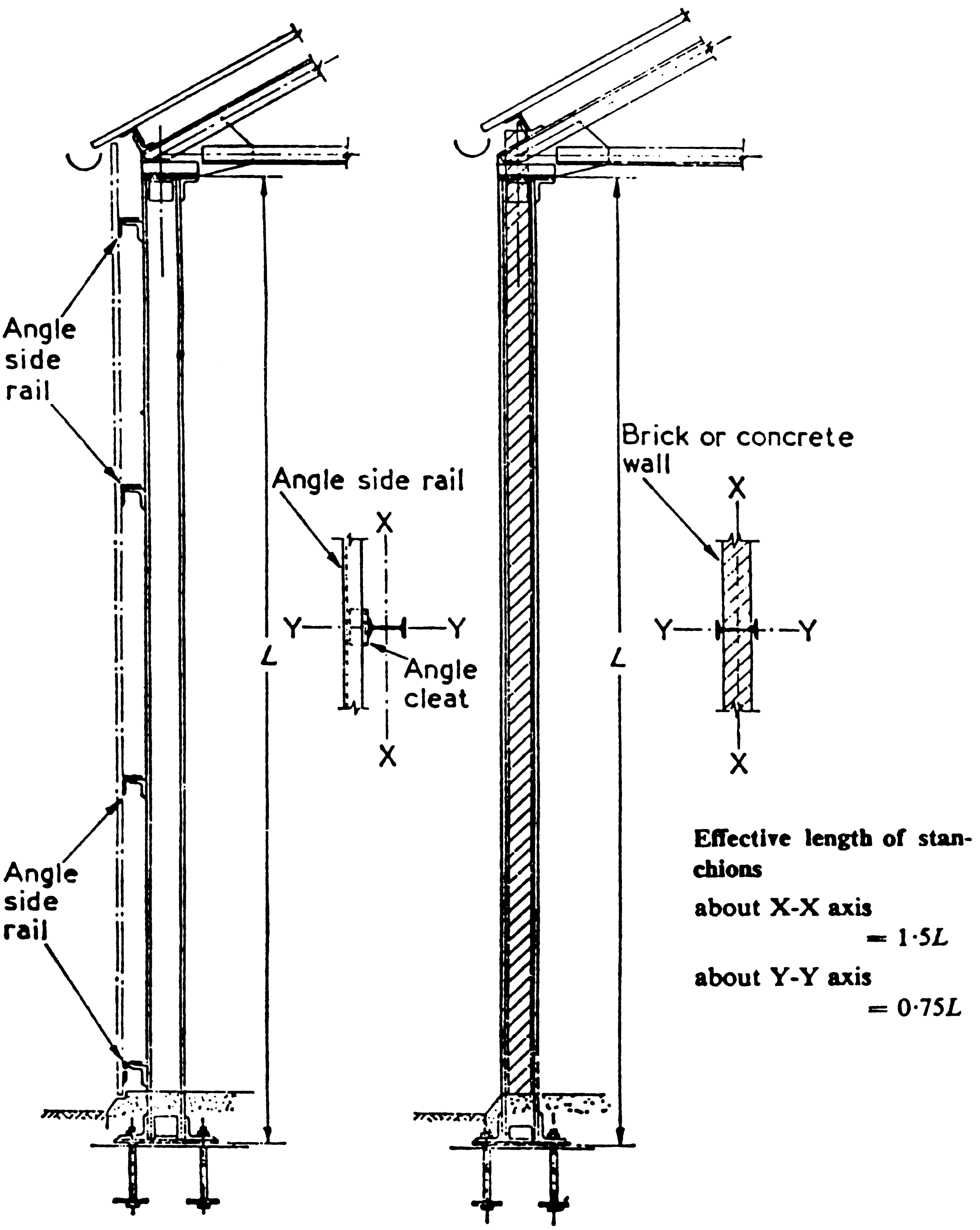


Figure 15 — Effective length of stanchions
Side stanchions in single storey building

Effective length of stanchion.

Axis X-X = 1·5*L*

Axis Y-Y = 0·75*L*$_1$
or 0·75*L*$_2$ whichever is the greater.

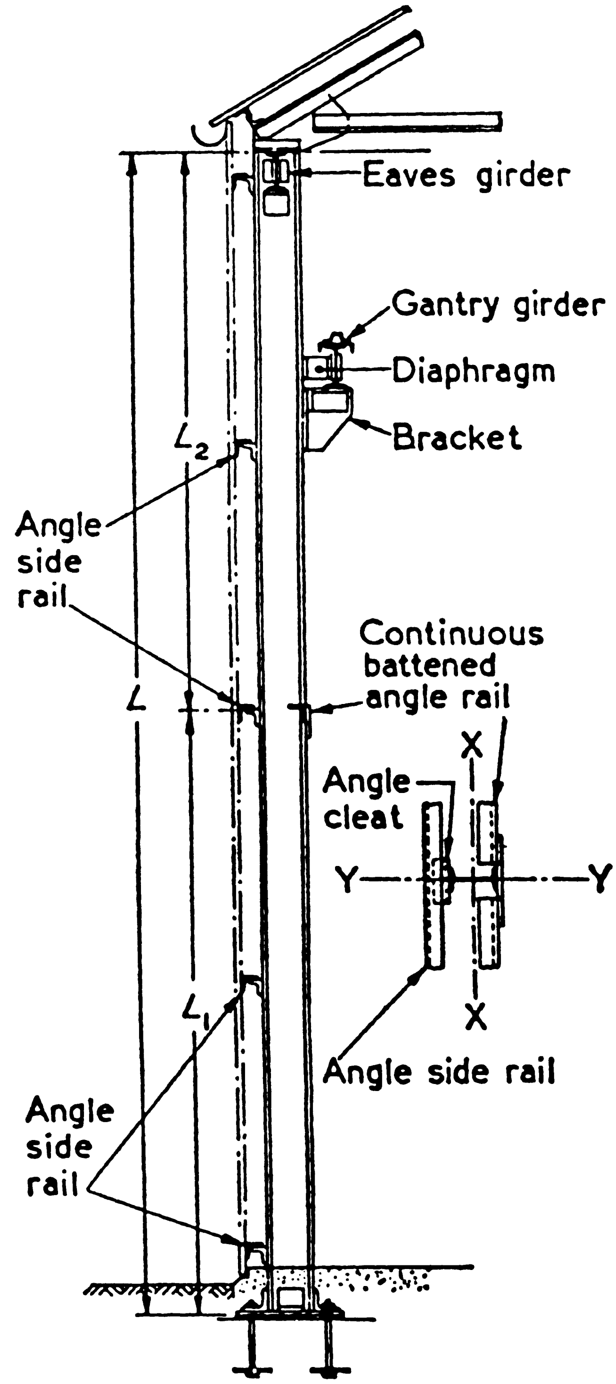


Figure 16 — Effective length of stanchions
Side stanchions in single storey building with crane gantry

Effective length of stanchion.

Axis X-X = 1·5L

Axis Y-Y = 0·75L_1

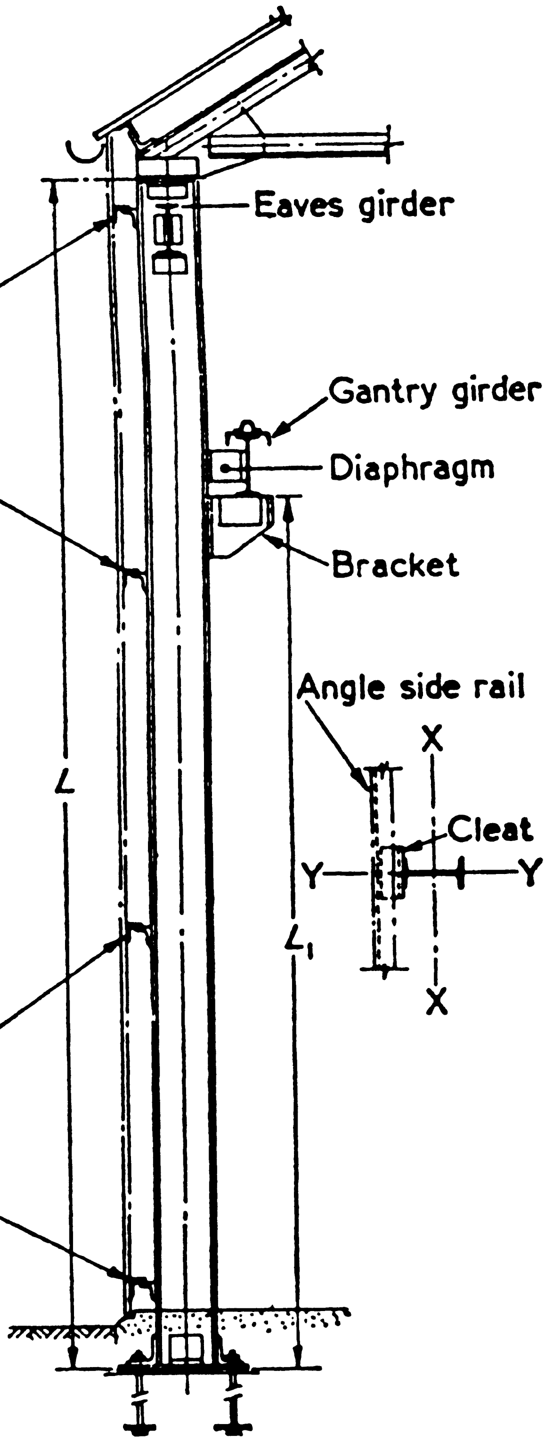


Figure 17 — Effective length of stanchions
Side stanchions in single storey building with crane gantry

Effective length of stanchions.

Roof stanchion R.

Axis X-X $= 1{\cdot}5L_1$

Axis B-B $= L_2, L_3, L_4$ or L_5 whichever is the greatest.

Crane stanchion C.

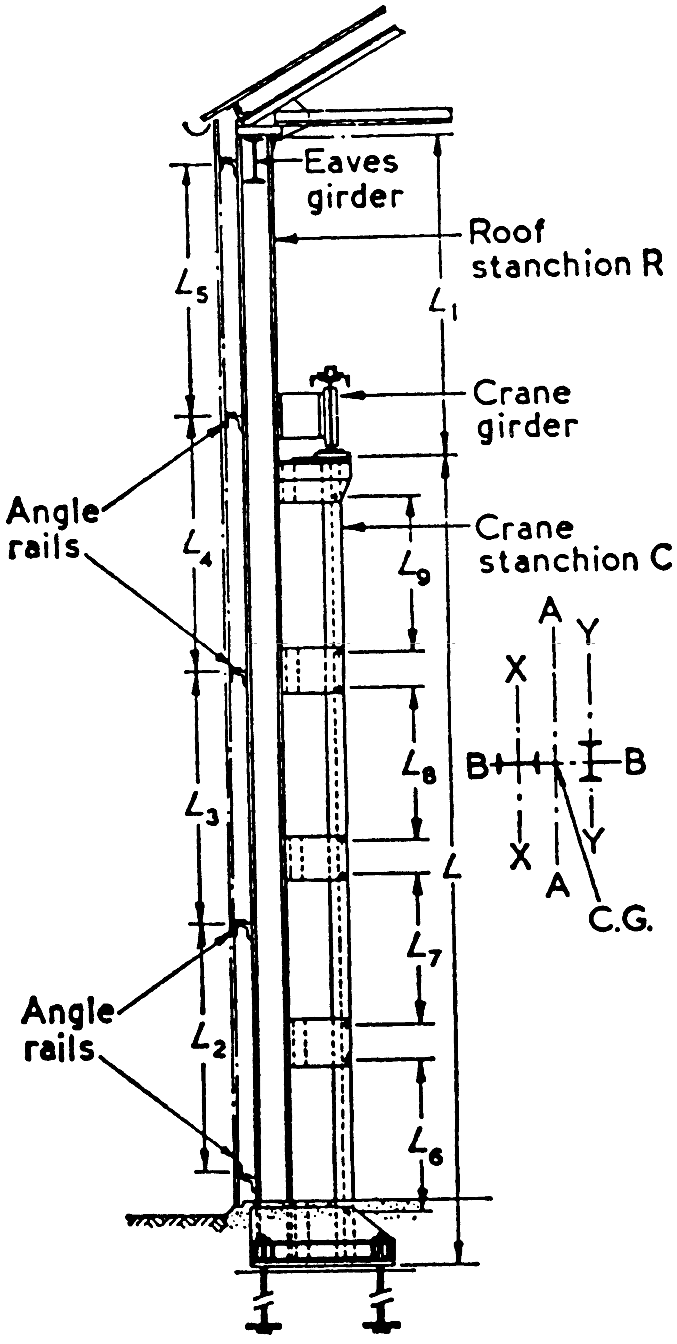
Axis B-B $= 0{\cdot}85L$

Axis Y-Y $= L_6, L_7, L_8$ or L_9 whichever is the greatest.

Combined roof stanchion R and crane stanchion C.

Axis A-A $= 1{\cdot}5L$

Axis B-B $= 0{\cdot}85L$



NOTE. Lengths L_6, L_7, L_8 and L_9 are the distances between the leading rivets in the batten plates or gussets.

Figure 18 — Effective length of stanchions
Side stanchions in single storey building with crane gantry

Effective length of stanchions.

Roof stanchion portion R_1.

Axis X-X $= 1{\cdot}5L_1$

Axis Y_2-Y_2 $= L_1$

Roof stanchion portion R.

Axis B-B $= 0{\cdot}85L$

Axis Y_1-Y_1 $= L_2$, L_3, L_4, L_5 or L_6 whichever is the greatest.

Crane stanchion C.

Axis B-B $= 0{\cdot}85L$

Axis Y-Y $= L_2$, L_3, L_4, L_5 or L_6 whichever is the greatest.

Combined roof stanchion portion R and crane stanchion C.

Axis A-A $= 1{\cdot}5L$

Axis B-B $= 0{\cdot}85L$

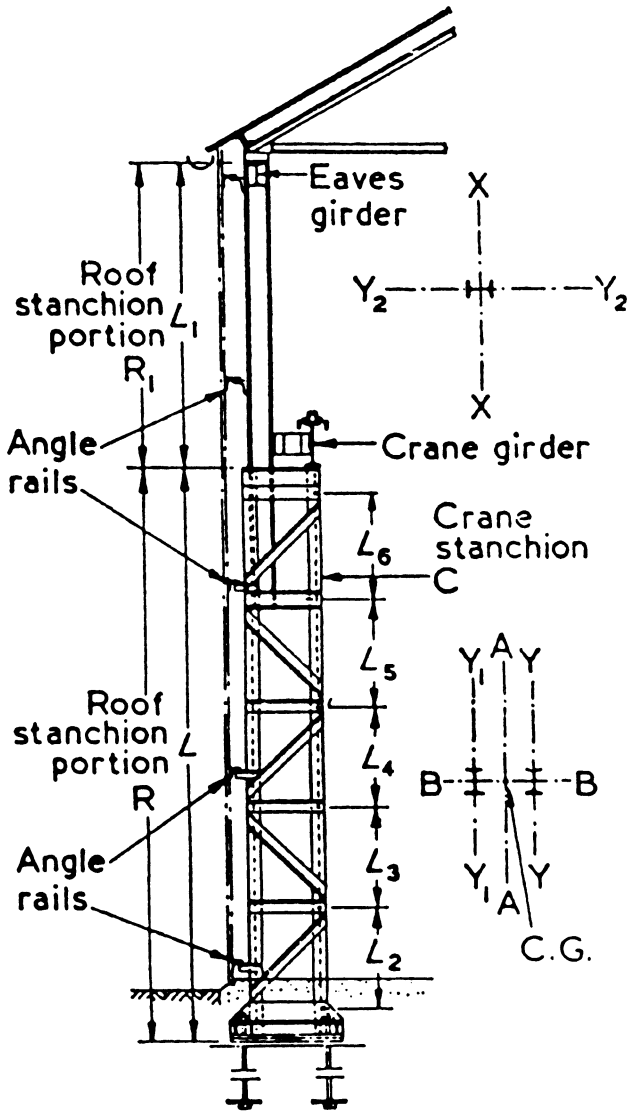


Figure 19 — Effective length of stanchions
Side stanchions in single storey building with crane gantry

Effective length of stanchions.

Roof stanchion R.

Axis B_1-B_1 = L_1

Axis A-A = $1{\cdot}5L_1$

For the separate channels about axis Y_1-Y_1 use L_7, L_8 or L_9 whichever is the greatest.

Crane stanchion C.

Axis B-B = $0{\cdot}85L$

Axis Y-Y = L_2, L_3, L_4, L_5 or L_6 whichever is the greatest.

Combined crane stanchion C.

Axis A-A = $1{\cdot}5L$

Axis B-B = $0{\cdot}85L$

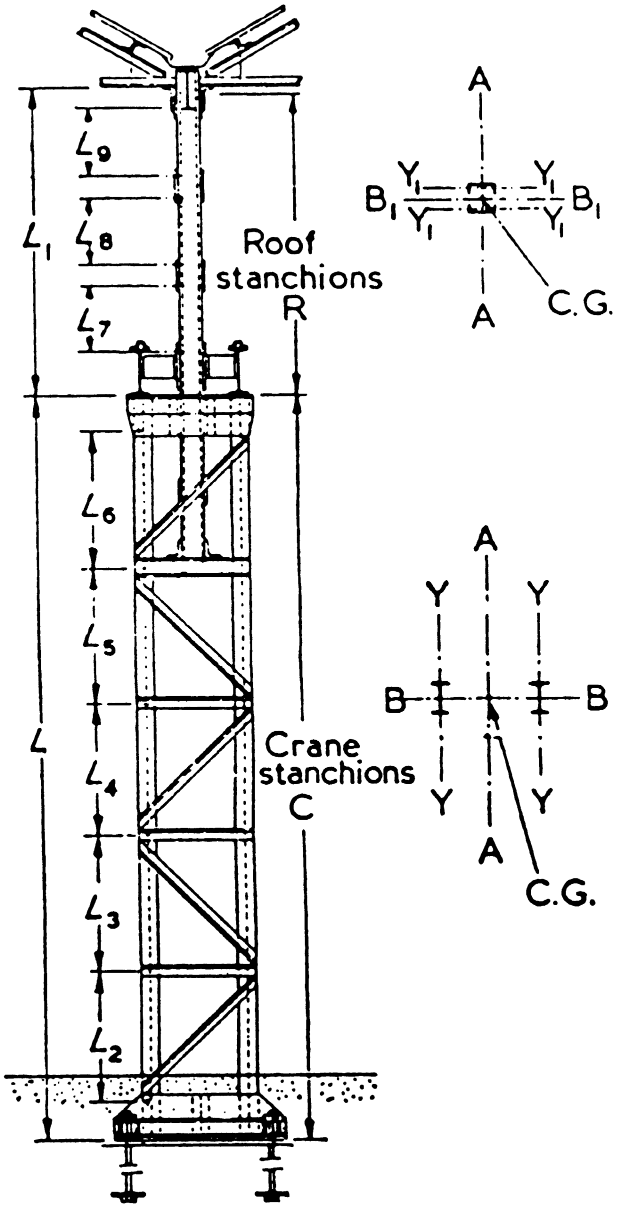


Figure 20 — Effective length of stanchions
Valley stanchion in single storey building with crane gantries

Appendix E General recommendations for steelwork tenders and contracts

The following recommendations are based on the former Code of Practice CP 113, "The structural use of steel in buildings", and are in line with those generally adopted for steelwork construction. They are given in this appendix for general information.

The recommendations in this appendix do not form part of the requirements of the standard and compliance with them is not necessary for the purpose of complying with B.S. 449.

The recommendations in this appendix are unsuitable for inclusion as a block requirement in a contract, but in drawing up a contract the points mentioned should be given consideration.

E.1 Exchange of information

Before the steelwork design is commenced, the building designer should be satisfied that the planning of the building, its dimensions and other principal factors, meet the requirements of the building owner and comply with the regulations of all authorities concerned. Collaboration of building designer and steelwork designer should begin at the outset of the project by joint consideration of the planning and such questions as the stanchion spacing, materials to be used for the construction, and depth of basement.

E.2 Information required by the steelwork designer

a) **General.**

i) Site plans showing in plan and elevation the proposed location and main dimensions of the building or structure.

ii) Ground levels, existing and proposed.

iii) Particulars of buildings or other obstructions which may have to remain on the actual site of the new building or structure during the erection of the steelwork.

iv) Particulars of adjacent buildings affecting, or affected by, the new work.

v) Stipulation regarding the erection sequence or time schedule.

vi) Conditions affecting the position or continuity of members.

vii) Limits of length and weight of steel members in transit and erection.

viii) Drawings of the substructure, proposed or existing, showing:

1) Levels of stanchion foundations, if already determined.

2) Any details affecting the stanchion bases or anchor bolts.

3) Permissible bearing pressure on the foundation.

4) Provisions for grouting, see Clause **76**.

In the case of new work, the substructure should be designed in accordance with the relevant codes dealing with foundations and substructure.

ix) The maximum wind velocity appropriate to the site. See CP 3:Chapter V, "Loading".

x) Reference to bye-laws and regulations affecting the steelwork design and construction.

b) **Further information relating to buildings.**

i) Plans of the floors and roof with principal dimensions, elevations and cross sections showing heights between floor levels.

ii) The occupancy of the floors and the positions of any special loads should be given.

iii) The building drawings, which should be fully dimensioned, should preferably be to scale of 1 to 100 and should show all stairs, fire-escapes, lifts, etc., suspended ceilings, flues, and ducts for heating and ventilating. Doors and windows should be shown, as the openings may be taken into account in the computation of dead load.

Requirements should be given in respect of any maximum depth of beams or minimum head room.

Large-scale details should be given of any special features affecting the steelwork.

iv) The inclusive weight per square metre of walls, floors, roofs, suspended ceilings, stairs and partitions, or particulars of their construction and finish for the computation of dead load.

The plans should indicate the floors which are to be designed to carry partitions. Where the layout of partitions is not known, or a given layout is liable to alteration, these facts should be specially noted so that allowance can be made for partitions in any position (see CP 3:Chapter V, "Loading").

v) The superimposed loads on the floors appropriate to the occupancy, as given in CP 3:Chapter V, "Loading", or as otherwise required.

Details of special loads from cranes, runways, tips, lifts, bunkers, tanks, plant and equipment.

vi) The grade of fire resistance appropriate to the occupancy, as given in CP 3:Chapter IV, "Precautions against fire", or as otherwise required.

E.3 Information required by tender (if not also the designer)

a) **General.**

i) All information listed under 2 *a*) above.

ii) Climatic condition at site — seasonal variations of temperature, humidity, wind velocity and direction.

iii) Nature of soil. Results of the investigation of sub-soils at site of building or structure.

iv) Accessibility of site and details of power supply.

v) Whether the steelwork contractor will be required to survey the site and set out or check, the building or structure lines, foundations and levels.

vi) Setting-out plan of foundations stanchions and levels of bases.

vii) Cross-sections and elevations of the steel structure, as necessary, with large-scale details of special features.

viii) Whether the connections are to be bolted, riveted or welded. Particular attention should be drawn to connections of a special nature such as turned bolts, high strength friction grip bolts, long rivets and overhead welds.

ix) Quality of steel, see Clause **3**, and provisions for identification.

x) Requirements in respect of protective paintings at works and on site; galvanizing or cement wash.

xi) Approximate dates for commencement and completion of erection.

xii) Details of any tests which have to be made during the course of erection or upon completion.

xiii) Schedule of quantities.

Where the tenderer is required to take off quantities, a list should be given of the principal items to be included in the schedule.

b) **Further information relating to buildings.**

i) Schedule of stanchions giving sizes, lengths and typical details of brackets, joints, etc.

ii) Plan of grillages, showing sizes, lengths and levels of grillage beams and particulars of any stiffeners required.

iii) Plans of floor beams showing sizes, levels, eccentricities and end moments. The beam reactions and details of the type of connection required should be shown on the plans.

iv) Plan of roof steelwork. For a flat roof the plan should give particulars similar to those of a floor plan. Where the roof is pitched, details should be given of trusses, portals, purlins, bracing, etc.

v) The steelwork drawings should preferably be to a scale of 1 to 100 and should give identification marks against all members.

vi) Particulars of holes required for services, pipes, machinery fixings, etc. Such holes should preferably be drilled at works.

E.4 Detailing

In addition to the number of copies of the approved drawings or details required under the contract, dimensioned shop drawings or details should be submitted in duplicate to the Engineer who should retain one copy and return the other to the steel suppliers or fabricators with any comments.

E.5 Time schedule

As the dates on which subsequent trades can commence depend on the progress of erection of the steel framing, the time schedule for the latter should be carefully drawn up and agreed by the parties concerned at a joint meeting.

E.6 Procedure on site

The steelwork contractor should be responsible for the positioning and levelling of all steelwork. Any checking or approval of the setting out by the general contractor or the Engineer should not relieve the steelwork contractor of his responsibilities in this respect.

E.7 Inspection

a) **Access to contractor's works.** The contractor should afford facilities for the inspection of the work at all stages.

b) **Inspection of fabrication.** Unless otherwise agreed this inspection should be carried out at the place of fabrication. The contractor should be responsible for the accuracy of the work and for any error which may be subsequently discovered.

c) **Inspection on site.** To facilitate inspection the contractor should, during all working hours, have a foreman or properly accredited charge hand available on the site, together with a complete set of contract drawings and any further drawings and instructions which may have been issued from time to time.

E.8 Maintenance

a) **General.** Where steelwork is to bc encased in solid concrete, brickwork or masonry, the question of maintenance should not arise, but where steelwork is to bc housed in hollow fire protection or is to be unprotected, particularly where the steelwork is exposed to a corroding agent, the question of painting or protective treatment of the steelwork should bc given careful consideration at the construction stage, having regard to the special circumstances of the case.

b) **Connections.** Where connections arc exposed to a corroding agent they should bc periodically inspected, and any corroded parts should bc thoroughly cleaned and painted.

Where bolted connections are not solidly encased and are subject to vibratory effects of machinery or plant, they should be periodically inspected and all bolts tightened.

List of references

This standard makes reference to the following British Standards and Codes of Practice.

BS 4, *Structural steel sections.*

BS 4-1, *Hot rolled sections.*

BS 499, *Welding terms and symbols.*

BS 592, *Carbon steel castings for general engineering purposes (included in BS 3100 — Steel castings for general engineering purposes).*

BS 639, *Covered electrodes for the manual metal-arc welding of carbon and carbon manganese steels.*

BS 1719, *Classification, coding and marking of covered electrodes for metal-arc welding.*

BS 1881, *Methods of testing concrete.*

BS 2573, *Rules for the design of cranes.*

BS 2573-1, *Specification classification, stress calculations and design criteria for structures.*

BS 2853, *Specification for the design and testing of steel overhead runway beams.*

BS 3100, *Specification for steel castings for general engineering purposes.*

BS 3692, *ISO metric precision hexagon bolts, screws and nuts.*

BS 4190, *ISO metric black hexagon bolts, screws and nuts.*

BS 4320, *Metal washers for general engineering purposes. Metric series.*

BS 4360, *Weldable structural steels.*

BS 4395, *High strength friction grip bolts and associated nuts and washers for structural engineering. Metric series.*

BS 4449, *Hot rolled steel bars for the reinforcement of concrete.*

BS 4482, *Hard drawn mild steel wire for the reinforcement of concrete.*

BS 4483, *Steel fabric for the reinforcement of concrete.*

BS 4604, *The use of high strength friction grip bolts in structural steelwork. Metric series.*

BS 4620, *Rivets for general engineering purposes. Metric series.*

BS 4870, *Approval testing of welding procedures.*

BS 4871, *Approval testing of welders working to approved welding procedures.*

BS 4872, *Approval testing of welders when welding procedure approval is not required.*

BS 4933, *ISO metric black cup and countersunk head bolts and screws with hexagon nuts.*

BS 5135, *Metal-arc welding of carbon and carbon-manganese steels.*

BS 6399, *Loading for buildings.*

PD 4064, *Addendum No.1 to BS 449-2:1969 The use of cold formed steel sections in building.*

CP 3, *Code of basic data for the design of buildings.*

CP 3-Chapter IV, *Precautions against fire.*

CP 3-Chapter V, *Loading.*

PD 3343, *Recommendations for design (Supplement No.1 to BS 449).*

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