

CODE OF PRACTICE
FOR THE STRUCTURAL USE OF STEEL
1987
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BUILDING AUTHORITY
HONG KONG

FOREWORD

This Code of Practice prepared under the direction of the Working Party on the review of the Building (Construction) Regulations provides for the design of structural steel on buildings.

It is intended that when further data on the application of plastic design, composite design and the use of cold formed sections in Hong Kong becomes available the scope of the code will be expanded.

A draft of this code was circulated to selected practising engineers, members of the construction industry and various Government Departments. All comments and views expressed have been taken into consideration in the preparation of this code now published.

It is acknowledged, that preparation of this Code owes a great deal to the time and effort freely given by Mr. C. K. Choi, Mr. G. Forrest-Brown, Mr. Y. C. Mok, Mr. B. G. Stevens, Mr. J. Thilwind and Mr. E. E. F. Taylor under the direction of Dr. R. C. T. Ho.

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1. GENERAL

1.1 SCOPE

This Code of Practice relates to the use in building works of hot rolled steel sections and plates and normalised tubular shapes.

The provisions of this Code of Practice are not deemed to apply to chimneys, bridges, docks piers or wharves, nor to structures designed on an experimental basis, except in so far as provided in 5.1.3 and Appendix A.

1.2 BRITISH STANDARDS AND CODES OF PRACTICE

Any reference to a British Standards Institution publication should be construed as follows:—

- (a) where a date is included in the reference, the reference is to the edition of that date, together with any amendments, supplements and addenda published at 30th June, 1986;
- (b) where no date is included in the reference, the reference is to the edition current at 30th June, 1986 together with any amendments, supplements and addenda published at that date;
- (c) any reference to any publication is a reference to so much only as is relevant in the context in which such a publication is quoted.

2. DEFINITIONS

For the purposes of this Code of Practice the following definitions shall apply:

BEAM or GIRDER	Any structural member which supports load primarily by internal resistance to bending.
DEAD LOAD	The weight of walls, floors, roofs, finishes, permanent partitions and other 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 Code of Practice.
FOUNDATION	That part of the building or structure in direct contact with and transmitting loads to the ground.
H-SECTION	A section with one central web and two plain flanges which has an overall depth not greater than 1.2 times the width of the narrower flange.
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.
HYBRID	Composed of elements of more than one strength grade of steel.
I-SECTION	A section with one central web and two plain flanges which has an overall depth greater than 1.2 times the width of the narrow flange.
IMPOSED LOAD	In respect of a building—all loads other than the dead load, but excluding wind load.
WIND LOAD	All loads due to the effects of wind pressure or suction.
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.
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, or column or other compression member.
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 specified in the appropriate material specification for the particular thickness of material.

2.1 NOTATION

The notations used in this Code of Practice have the following meanings with respect to the structure, member or condition to which the clause is applied, unless otherwise defined elsewhere in this Code.

b_1 is the outstand of a flange or stiffener beyond the line of connections to a web or other line of support

b_2 is the width of a flange between two adjacent lines of connections to other parts of a member, or other line of support

C_o is Eulers critical stress

D is the overall depth of a section measured parallel to the web

d_1 is (1) for the web of a beam without horizontal stiffeners—the clear distance between flanges, neglecting fillets, or the clear distance between inner toes of the flange angles as appropriate

or (2) for the web of a beam with horizontal stiffeners—the clear distance between the horizontal stiffeners and the tension flange, neglecting fillets, or the clear distance between inner toes of the tension flange angles, as appropriate

d_2 is twice the clear distance from the neutral axis of a beam to the compression flange neglecting fillets or the inner toes of the flange angles, as appropriate

d_3 is the clear distance of web between root fillets

E is the Modulus of Elasticity for steel, taken as 2.0×10^5 MPa in this Code

f_{bc} is the calculated compressive stress in a member due to bending about a principal axis

f_{bt} is the calculated tensile stress in a member due to bending about a principal axis

f_c is the calculated average stress in a member due to an axial compressive force

f_e is the calculated equivalent stress in a member due to an axial compressive force

f_t is the calculated average stress in a member due to an axial tensile force

F_{ob} is the elastic buckling stress of a beam

L is the effective length

p_b is the maximum allowable stress due to bending

p_{bc} is the maximum allowable compressive stress due to bending in a member not subject to an axial force

p_{br} is the maximum allowable bearing stress

p_{bt} is the maximum allowable tensile stress due to bending in a member not subject to axial force

p_c is the maximum allowable compressive stress in axial loaded members

p_e is the maximum allowable equivalent stress due to the action of combined loadings

p_q is the maximum allowable shear stress in a member

p_{qv} is the maximum allowable average shear stress in a member

p_t is the maximum allowable tensile stress in an axially loaded tensile member

r_y is the radius of gyration of a section about its minor axis

t is the thickness of the web of a section

T is the effective thickness of the flange in a flanged section

T_b is the stiffness of a braced structure

T_u is the stiffness of the structure with the bracing system removed

Y_s is the yield stress in tension for steel, in MPa

3. MATERIALS

3.1 STRUCTURAL STEEL

Structural steel should be of one of the following grades:

Grade 250—refers to a grade of steel with a nominal yield strength of 250 MPa and having similar chemical composition and mechanical properties to these specified in BS 4360 for Grade 43 steel.

Grade 350—refers to a grade of steel with a nominal yield strength of 350 MPa and having similar chemical composition and mechanical properties to these specified in BS 4360 for Grade 50 steel.

Grade 450—refers to a grade of steel with a nominal yield strength of 450 MPa and having similar chemical composition and mechanical properties to these specified in BS 4360 for Grade 55 steel.

and should be accompanied by Mill Certificates showing that the requirements of chemical composition and mechanical properties of BS 4360 are satisfied.

3.2 *ELECTRODES*

Electrodes should comply with the requirements of BS 639.

3.3 *BOLTS AND NUTS*

Bolts and nuts should comply with one of the following:

BS 3692
BS 4190
BS 4933

3.4 *WASHERS*

Plain washers should be made of steel. Taper or other specially shaped washers should be made of steel or malleable cast iron. (refer BS 4320)

3.5 *HIGH STRENGTH FRICTION GRIP BOLTS*

High strength friction grip bolts should conform to BS 4395: Parts 1 and 2 and their use should conform to BS 4604: Parts 1, 2 and 3.

4. **LOADS**

4.1 *DEAD, WIND AND IMPOSED LOADS*

The determination of dead loads, wind loads, and imposed loads should be in accordance with the Building (Construction) Regulations.

4.2 *DYNAMIC LOADS*

Where loads arising from machinery, runways, cranes and other plant producing dynamic effects are supported by or communicated to the building or part of the building, the forces produced by dynamic effects should be considered as additional imposed loads in the design of the building. In order to ensure due economy in design the Engineer should ascertain as accurately as possible the appropriate dynamic increase for all members affected.

For crane gantries, with a lifting capacity of less than 10 tonne, the following allowances may be deemed to cover all forces set up by vibration, shock from slipping of slings, kinetic action of acceleration and retardation and impact of wheel loads:

4.2.1 For loads acting vertically, the maximum static wheel loads should be increased by the following percentages:

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

4.2.2 The horizontal force acting transverse to the rails should 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 should be taken into account when considering the lateral rigidity of the rails and their fastenings.

4.2.3 Horizontal forces acting along the rails should 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
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The forces specified in either 4.2.2 or 4.2.3 above should be considered as acting at the rail level and being appropriately transmitted to the supporting systems.

Gantry girders and their vertical supports should be designed on the assumption that either of the horizontal forces specified in 4.2.2 or 4.2.3 may act at the same time as the vertical load.

An increase of 10 per cent on the permissible stresses specified in this Code of Practice may be allowed for the combination of loadings specified in 4.2.1 and 4.2.2 above in respect of the design of the gantry girders and supporting structures. This increase is not however in addition to that permitted in 5.4.

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.

Structures or structural components which support hoisting devices with a lifting capacity of 10 tonnes or more, should be designed and constructed in accordance with the relevant provisions of BS 2573: Part 1.

In the event that BS 2573:Part 1 does not adequately cover any particular situation then the structure or structural component should be designed to the satisfaction of the Building Authority.

Note: For factors applicable to the allowable working stresses, and detailed design under fatigue conditions, see BS 153:Parts 3B and 4 and BS 5400:Part 10.

4.3 COMBINED WIND AND DYNAMIC LOADS

A reduction in wind loading may be allowed where the operation of three or more overhead cranes is to be provided in a structure. The supporting structure may be designed for the following load cases:

- (1) the loads due to the operation of the single heaviest crane should be combined with the full wind load appropriate to the height of the structure.
- (2) the loads due to the worst combination of crane loads should be combined with 60 per cent of the wind load appropriate to the height of the structure.

5. DESIGN AND DETAILS OF CONSTRUCTION

5.1 GENERAL

5.1.1 STEEL FRAMEWORK

Any part of a structure should be capable of sustaining the most adverse combination of static and dynamic forces which may reasonably be expected from dead, wind and imposed loads referred to in the Building (Construction) Regulations without the permissible stresses specified in this Code of Practice being exceeded.

Frames may be categorised as braced or unbraced depending on their mode of resistance to lateral load effects. If special structural elements such as shear walls, core walls, truss members etc., are used to resist the lateral loads then the frame is said to be *braced*. It is said to be *unbraced* if only the flexural rigidity of the members is relied upon to provide the lateral resistance.

For a bracing system to be effective the following relationship should be satisfied:

$$T_b/T_u > 5$$

Where T_b is the stiffness of the braced structure, and

T_u is the stiffness of the structure with the bracing system removed.

If this condition is not fulfilled then the structure must be considered as unbraced.

5.1.2 VERTICAL LOAD—LATERAL DEFLECTION EFFECTS

Where a building is sensitive to the effects of lateral deflection the Engineer should take into account the additional actions induced by the lateral displacements of the points of application of the vertical loads. (P— Δ effects)

5.1.3 DESIGN OF FRAMEWORKS

Steel frameworks may be designed using the following methods:

(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 purposes of design, be assumed to be pin-jointed.

This method involves the following assumptions:

- (a) beams are simply supported,
- (b) all connections of beams girders or trusses are proportioned to resist the shear forces applied at the appropriate eccentricity,

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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 purposes of design, be assumed to be pin-jointed.

This method involves the following assumptions:

- (a) beams are simply supported,
- (b) all connections of beams girders or trusses are proportioned to resist the shear forces applied at the appropriate eccentricity,

- (c) members in compression are subjected to loads applied at the appropriate eccentricities (refer 7.5 for stanchions), and effective lengths (refer 7.2),
- (d) members in tension are subjected to longitudinal loads applied over the net area of the section (refer to 8.2).

(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 should be made to show that the stresses in any part of the structure are not in excess of those laid down in this Code of Practice.

Alternatively 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.
- (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 7.1.2 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 should be carried out in accordance with accurate methods of elastic analysis and to the limiting stresses permitted in this Code of Practice.

5.1.4 EXPERIMENTAL BASIS

Where by reason of the unconventional nature of construction, calculation is not practicable, or where the methods of design given above are inapplicable or inappropriate, loading tests should 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, imposed and wind loads, and
- (2) adequate stiffness to resist, without excessive deflection, a total load equal to the sum of the dead load and 1.5 times the imposed and wind loads and, in the case of beams, to satisfy 5.6.

5.1.5 CURVED MEMBERS

The design of curved or bent members should be given special consideration. Allowance should be made for any thinning of the bent part which may be caused by bending the member.

5.2 RESISTANCE TO LATERAL FORCES

Adequate provision should be made to resist the lateral forces that can occur during and after erection of a structure. Where lateral forces cause twisting in a structural frame, provision should be made to resist all the resulting increases in horizontal shear but all resulting decreases in horizontal shear should be neglected in the design.

Where high speed travelling cranes are supported by the structure or where a structure may be otherwise subject to vibration or sway then a bracing system should be provided to reduce the vibration or sway to a suitable minimum.

When considering the effect of the lateral forces, proper allowance should be made for the strength and stiffness of all structural components as follows:

- (1) where structural components such as walls, roofs, floors and other additional bracing structure are capable of transmitting all lateral forces to the substructure, it may be deemed that the structural steelwork is not loaded by such forces.

- (2) where these other structural components are strong enough to transmit only part of the lateral forces to the substructure, the lateral forces should be distributed through the structural system in accordance with the relative stiffnesses of the frame and other components, and the structural steelwork should be designed accordingly.
- (3) where resistance to lateral forces provided by other structural components is obviously low or not ascertainable, the structural steelwork should be designed to resist all the lateral forces and transmit them to the substructure.

5.3 MINIMUM THICKNESS OF METAL

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

- 5.3.1 steel used for external construction exposed to the weather or other corrosive influences should 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 standard rolled steel joists and channels or packings.
- 5.3.2 sealed tubes or sealed box sections used for external construction exposed to the weather should have walls not less than 4 mm thick and for construction not so exposed should have walls not less than 3 mm thick.

5.4 STRESSES IN FRAMES DUE TO WIND FORCES

Unless otherwise stated, the allowable stresses given in this Code of Practice 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 is not less than that needed if the wind stresses were neglected. This provision does not apply to grillage beams (refer 7.11).

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 5.5.1 and p_t and p_{bt} in 5.5.2 should be increased by 25 per cent in satisfying the relationship to unity, provided that 5.5.1 and 5.5.2 shall also apply in respect of any combination of stresses arising from bending and axial loading not due to wind.

5.5 COMBINED STRESSES

5.5.1 BENDING AND AXIAL COMPRESSION

Members subject to both axial compression and bending should be so proportioned that the following requirements are satisfied

- (1) For the member as a whole and using the maximum value of f_{bc} :

$$\frac{f_c}{p_c} + \frac{C_m f_{bc}}{(1 - \frac{f_c}{0.6C_o}) p_{bc}} \leq 1.0$$

However where f_c/p_c is less than 0.15, the following expression may be used:

$$\frac{f_c}{p_c} + \frac{f_{bc}}{p_{bc}} \leq 1.0$$

The value of p_{bc} to be used in the above formulas should be the lesser of the values of the maximum permissible stresses, p_{bc} and p_b , given in 6.2.2 and 6.3.3 respectively for bending and about the appropriate axis.

- (2) At a support and using the value of f_{bc} at the support:

$$\frac{f_c}{0.6 Y_s} + \frac{f_{bc}}{p_{bc}} \leq 1.0$$

In cased struts where allowance is made for the load carried by the concrete in accordance with 7.1.2, the ratio f_c/p_{bc} in the above expressions should be replaced by the ratio of the calculated axial load on the strut to the allowable axial load determined from 7.1.2.

In the above expressions the following extra notations are used:

$$(a) \quad C_o = \frac{\pi^2 E}{(L/r)^2}$$

where L is the effective length in the plane of bending;

r is the corresponding radius of gyration.

(b) C_m —a coefficient whose value should be taken as follows:—

(i) For members in frames where sidesway is not prevented:

$$C_m = 0.85$$

(ii) For members in frames where sidesway is prevented and not subject to transverse loading between their supports in the plane of bending:

$$C_m = 0.6 - 0.4\beta$$

but not less than 0.4

where β is the ratio of the smaller to the larger moments at the ends of that portion of the unbraced member in the plane of bending under consideration;

β is positive when the member is bent in reverse curvature and negative when bent in single curvature.

(c) For members in frames where sidesway is prevented in the plane of loading and subjected to transverse loading between their supports, the value of C_m may be determined by rational analysis. In the absence of such an analysis the following values may be used

for members whose ends are restrained $C_m = 0.85$

for members whose ends are unrestrained $C_m = 1.00$

5.5.2 BENDING AND AXIAL TENSION

Members subject to both axial tension and bending stresses should be so proportioned that the quantity

$$\frac{f_t}{p_t} + \frac{f_{bt}}{p_{bt}} \leq 1 \text{ is satisfied,}$$

Where f_t is the calculated axial tensile stress,

p_t is the allowable axial tensile stress,

f_{bt} is the resultant tensile stress due to bending about both principle axes;

p_{bt} is the appropriate allowable tensile stress in bending.

5.5.3 BENDING AND SHEAR

The equivalent stress f_e due to bending and shear should not exceed the value given by:

$$f_e = Y_s / 1.1$$

Where Y_s is the specified yield stress.

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.

5.5.4 COMBINED BEARING, BENDING AND SHEAR

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, should not exceed the values of p_e given in Table 1.

$$f_e = \sqrt{(f_{bt}^2 + f_{br}^2 + f_{bt} f_{br} + 3 f_q^2)}$$

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

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

Table 1. Allowable equivalent stress, p_e

Steel Grade	Material Thickness	p_e
250	Up to and including 40 mm	$Y_s/1.1$
	over 40 mm	$Y_s/1.19$
350	all	$Y_s/1.10$
450	up to and including 40 mm	$Y_s/1.16$
	over 40 mm	$Y_s/1.25$

Note: The increase permitted by 4.2 and 5.1 do not apply to these values

5.6 DEFLECTION OF BEAMS

Under the most adverse loading conditions, the deflections of neither a structure as a whole, nor any of its parts, should be such as will impair the strength or serviceability of the structure or part, or lead to damage of other building components, or be unsightly.

Note: Guidance on maximum permissible deflections is given in Appendix B.

5.7 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 should 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.

5.8 SECTIONAL AREAS

5.8.1 The gross sectional area should be taken as the area of the cross section as calculated from the specified size.

The net sectional area should be taken as the gross sectional area less deductions for bolt holes and open holes, or other deductions specified in this Code of Practice.

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

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

For (1) all axially loaded tension members;
(2) plate girders with d_1/t greater than $1440/\sqrt{Y_s}$

Where t is the thickness of the web,

d_1 is the depth, defined as the clear distance between flange angles or, where there are no flange angles, the clear distance between flanges, ignoring fillets. Where tongue plates having a thickness of not less than twice the thickness of the web plate are used, the depth d_1 should be taken as the depth of the girder between the flanges less the sum of the depths of the tongue plates;

Y_s is the specified yield stress.

the area to be deducted when the holes are staggered should 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 $s^2 t_1 / 4G$ for each gauge space in the chain of holes.

Where s is 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 is the thickness of the holed material;

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

For sections such as angles with holes in both legs the gauge should 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 bolts joining the parts between such chains of holes should be taken into account in determining the strength of the member.

- 5.8.2 The net sectional area of a bolt or screwed tension rod with ISO metric threads should be taken as the tensile stress area of the threaded part or the cross-sectional area of the unthreaded part, whichever is the lesser.

5.9 SEPARATORS AND DIAPHRAGMS

Where two or more rolled steel joists or channels are used side by side to form a girder, they should 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 another or are required to be distributed between beams, diaphragms and their fastenings designed with sufficient stiffness to distribute the load should 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, should be used.

6. DESIGN OF MEMBERS SUBJECT TO BENDING

6.1 GENERAL

The bending stress in the extreme fibres of a member should not exceed any of the appropriate maximum permissible stresses given in 6.2, 6.3, 6.4 and 6.5 for bending, in 6.6 for bearing and 6.7 for shear, except that 6.3 should not apply in the following cases:

- (1) a beam bent about its axis of minimum strength,
- (2) an I-beam or channel with equal flanges and with a value of L/r_y less than $940/\sqrt{Y_s}$;
- (3) a rectangular hollow section beam with B/D and t/T both greater than 0.25.

where B is the overall width of section,

D is the overall depth of section,

t is the web thickness;

T is the flange thickness.

6.2 MAXIMUM PERMISSIBLE STRESSES

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

6.2.1 FOR PARTS IN TENSION

the maximum permissible stress p_{bt} determined by:

$$p_{bt} = 0.66 Y_s$$

for material thickness up to 40 mm, or

$$p_{bt} = 0.60 Y_s$$

for material thickness greater than 40 mm.

6.2.2 FOR PARTS IN COMPRESSION

the maximum permissible compression stress in bending p_{bc} , which should be the lesser of the value determined by the following formulae as appropriate, but not greater than $0.6 Y_s$.

$$p_{bc} = \left[0.72 - \frac{0.12}{256} \frac{b_1}{T_1} \sqrt{Y_s} \right] Y_s$$

$$p_{bc} = \left[0.72 - \frac{0.12}{F_a} \frac{b_2}{T_1} \sqrt{Y_s} \right] Y_s$$

For sections such as angles with holes in both legs the gauge should 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 bolts joining the parts between such chains of holes should be taken into account in determining the strength of the member.

- 5.8.2 The net sectional area of a bolt or screwed tension rod with ISO metric threads should be taken as the tensile stress area of the threaded part or the cross-sectional area of the unthreaded part, whichever is the lesser.

5.9 SEPARATORS AND DIAPHRAGMS

Where two or more rolled steel joists or channels are used side by side to form a girder, they should 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 another or are required to be distributed between beams, diaphragms and their fastenings designed with sufficient stiffness to distribute the load should 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, should be used.

6. DESIGN OF MEMBERS SUBJECT TO BENDING

6.1 GENERAL

The bending stress in the extreme fibres of a member should not exceed any of the appropriate maximum permissible stresses given in 6.2, 6.3, 6.4 and 6.5 for bending, in 6.6 for bearing and 6.7 for shear, except that 6.3 should not apply in the following cases:

- (1) a beam bent about its axis of minimum strength,
- (2) an I-beam or channel with equal flanges and with a value of L/r_y less than $940/\sqrt{Y_s}$;
- (3) a rectangular hollow section beam with B/D and t/T both greater than 0.25.

where B is the overall width of section,

D is the overall depth of section,

t is the web thickness;

T is the flange thickness.

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

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the maximum permissible stress p_{bt} determined by:

$$p_{bt} = 0.66 Y_s$$

for material thickness up to 40 mm, or

$$p_{bt} = 0.60 Y_s$$

for material thickness greater than 40 mm.

6.2.2 FOR PARTS IN COMPRESSION

the maximum permissible compression stress in bending p_{bc} , which should be the lesser of the value determined by the following formulae as appropriate, but not greater than $0.6 Y_s$.

$$p_{bc} = \left[0.72 - \frac{0.12}{256} \frac{b_1}{T_1} \sqrt{Y_s} \right] Y_s$$

$$p_{bc} = \left[0.72 - \frac{0.12}{F_a} \frac{b_2}{T_1} \sqrt{Y_s} \right] Y_s$$

Where $F_a = 560$ for welded flanges or plates which are not stress relieved,

$F_a = 800$ for other flanges or plates,

b_1 is the outstand of the flange or plate of thickness T_1 in compression,

b_2 is the unsupported width of the flange or plate of thickness T_1 in compression;

Y_s is the specified yield stress.

6.3 MAXIMUM PERMISSIBLE STRESSES IN A BEAM BENT ABOUT ITS AXIS OF MAXIMUM STRENGTH

6.3.1 EQUAL FLANGE I-BEAMS OR CHANNELS:

For an I-beam or channel with equal flanges of uniform cross section throughout bent about the axis of maximum strength, the bending stress should not exceed the maximum permissible stress p_b calculated from the formulae in 6.3.3 as appropriate with $F_{ob} = A$ MPa, where A is defined in 6.4.

6.3.2 LATERALLY UNSUPPORTED ANGLE SECTIONS

For laterally unsupported angle sections, the maximum permissible stress p_b may be calculated from the formulae with $K_2 = -1$ and an elastic flexural-torsional analysis.

6.3.3 OTHER SECTIONS

The maximum calculated stress should not exceed the maximum permissible stress given by:

$$p_b = \left[0.55 - \frac{0.10 F_{ob}}{Y_s} \right] F_{ob}$$

for case where $F_{ob} \leq Y_s$; or

$$p_b = \left[0.95 - 0.5 \sqrt{\frac{Y_s}{F_{ob}}} \right] Y_s$$

for the case where $F_{ob} > Y_s$

as appropriate, where the maximum stress F_{ob} in the beam at elastic buckling should be calculated in accordance with 6.4 or by an elastic flexural-torsional buckling analysis.

6.4 ELASTIC CRITICAL STRESS

If an elastic flexural-torsional buckling analysis is not carried out, the elastic critical stress F_{ob} should be determined by:

$$F_{ob} = K_1(A + K_2B)C_2/C_1 \text{ MPa}$$

For an I-beam with values of T/t not greater than 2.0 and d_1/t not greater than $1344/\sqrt{Y_s}$, the value of F_{ob} should be multiplied by a factor of 1.20.

Where $B = \left(\frac{1675}{L/r_y} \right)^2$; and

$$A = B \sqrt{1 + 0.05 \left(\frac{L}{r_y} \frac{T}{D} \right)^2}$$

For beams with varying value of r_y the value of r_y at the point of maximum moment should be used.

C_1 , C_2 are respectively the lesser and the greater distances from the section neutral axis to the extreme fibres.

K_1 is a coefficient to allow for the reduction in thickness or breadth of the flanges between points of effective lateral restraint. Values of K_1 are given by:

$$K_1 = N + 0.2$$

but not greater than 1.0

Where the value of N calculated for the compression flange alone is smaller than that when both flanges are combined, this smaller value of N should be used,

N is the ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between points of effective lateral restraint. The minimum value of N should be 0.25 and the flange reductions should be limited accordingly. The removal of a flange at the end of a beam should be neglected when calculating N, provided that the length of the flange removed does not exceed a length equal to the flange width at midspan,

K_2 is a coefficient to allow for the inequality of the flanges;

if $M < 0.5$ then $K_2 = 2 (M - 0.5)$

if $M \geq 0.5$ then $K_2 = M - 0.5$

M is the ratio of the moment of inertia of the compression flange alone to the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the axis of minimum strength of the beam, at the point of maximum bending moment.

6.5 BENDING STRESSES (CASED BEAMS)

6.5.1 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 encased in concrete, with 10 mm aggregate (unless solidity can be obtained with larger aggregate), and not lower than grade 20.
- (3) the minimum width of casing is equal to $(B + 100)$ mm where B is the overall width of the steel flange or flanges in millimetres.
- (4) the outer faces and edges of the beam have a concrete cover of not less than 50 mm.
- (5) the casing is effectively reinforced with either:
 - (a) hard drawn wire conforming to BS 4482, or
 - (b) steel fabric conforming to BS 4483, or
 - (c) steel bars conforming to BS 4449.

The wire shall be at least 5 mm diameter and the reinforcement should be in the form of links 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.

6.5.2 The maximum calculated bending stress should not exceed the lesser of:

- (1) the value of any appropriate maximum permissible stresses permitted by 6.2, 6.3, and 6.4 in which the radius of gyration r_y may be taken as $0.2(B + 100)$ mm, and D/T should be as for the uncased section, and
- (2) 1.5 times that permitted for the uncased section.

Note: This does not apply to box sections.

6.6 BEARING STRESS

The calculated bearing stress on the net projected area of contact should not exceed the value of p_{br} given by:

$$p_{br} = 0.75 Y_s$$

6.7 SHEAR STRESS

6.7.1 MAXIMUM PERMISSIBLE SHEAR STRESS

The maximum shear stress, calculated in accordance with the distribution of stresses in conformity with the elastic behaviour of the members in flexure and shear should not exceed p_q given by:

$$p_q = 0.45 Y_s$$

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

6.7.2 AVERAGE SHEAR STRESS IN ROLLED I-BEAMS CHANNELS, PLATE GIRDERS, BOX SECTIONS, RECTANGULAR AND CIRCULAR HOLLOW SECTIONS

The average shear stress on the effective sectional area should not exceed the value P_{qv} given by:

- (1) For unstiffened web

$$p_{qv} = 0.37 Y_s$$

- (2) For stiffened web

$$p_{qv} = 0.37 Y_s \left(1.3 - \frac{\sqrt{Y_s} \frac{b}{t}}{4000 [1 + 0.5 (\frac{b}{a})^2]} \right)$$

Where a is the greater clear dimension of the web panel;

b is the lesser clear dimension of the web panel.

The effective sectional area should be taken as:

- (a) *rolled I-beams and channels*:—the product of the thickness of the web and the full depth of the section.
- (b) *plate girders and box girders*:—the product of the thickness of the web or webs and the full depth of the web plate.
- (c) *circular hollow sections*:—60 per cent of the gross sectional area.

Compliance with 6.7.2 may be deemed to satisfy the requirements of 6.7.1 above.

For webs which have tongue plates or which are reinforced by additional plates, the maximum shear should be calculated and the beam designed so as to satisfy both 6.7.1 and 6.7.2.

Note: The allowable stresses in 6.7.1 apply provided any reduction of the web cross sectional area is only due to bolt holes etc. Where large apertures are cut in the web a special analysis should be made to ensure that the stresses given in this Code of Practice are not exceeded.

6.8 EFFECTIVE SPAN OF BEAMS

The effective span of a beam should be taken to be the distance between the centres of supports, except for the provision of 5.1.3(3) and where, under 7.5.1(1) and 7.5.1(2) the point of application of the reaction is taken as eccentric to the support, when it should be permissible to take the effective span as the length between the assumed points of application of the reactions.

6.8.1 SLENDERNESS RATIO OF COMPRESSION FLANGES

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

6.9 EFFECTIVE LENGTH OF COMPRESSION FLANGES FOR BEAMS AND GIRDERS

- 6.9.1 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 6.2, 6.3 and 6.5 should be taken as follows:

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

$$L = \text{span}$$

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

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

- (3) with ends of the compression flange 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, or their being built into walls.

Where the ends of the beam are not restrained against torsion, or where the load is applied to the compression flange and both the load and the compression flange are free to move laterally, the above values of the effective length should be increased by 20 per cent.

Note: The end restraint element should 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 flange and having a value equal to not less than 2.5 per cent of the maximum force occurring in the flange.

- 6.9.2 For beams which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in 6.9.1(1) above, the effective length of the compression flange should be taken as the maximum distance, centre to centre, of the restraint members.
- 6.9.3 When the projecting length of a cantilever has no intermediate lateral restraint, then, except when moment is applied at the tip, the notional effective lengths given in Table 2 should be used. The de-stabilising loading condition applies whenever load is applied to the top flange of the cantilever and both this load and the flange are free to move laterally.
- 6.9.4 Where beams support slab construction, the beam may 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 equal to 2.5 per cent of the maximum force in the compression flange of the beam, considered as distributed uniformly along the flange. Furthermore, the slab construction should be capable of resisting this lateral force in lateral flexure and shear.
- 6.9.5 For beams which are provided with members giving effective lateral restraint of the compression flange at intervals along the span, the effective restraint should be capable of resisting a force equal to 2.5 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.

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 should be taken as 2.5 per cent of the maximum flange force in one beam only.

In the case 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 should be taken as 2.5 per cent of the maximum force in the compression flange plus 1.25 per cent of this force for every of the series other than the first up to a maximum total of 7.5 per cent.

6.10 BEAMS WITH SOLID WEBS INCLUDING PLATE GIRDERS

6.10.1 SECTIONAL PROPERTIES

- (1) 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 area should be increased in the ratio of gross area to the effective area of the flange section. For this purpose the flange sectional area in bolted construction should 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 should be taken as that of the flange plates plus that of the tongue plates (if any) up to a limit of eight times their thickness, which should be not less than twice the thickness of the web.
- (2) The effective sectional area of compression flanges should be the gross area with the deductions for excessive width of plates as specified for compression members in 7.3 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.
- (3) The effective sectional area of the tension flanges should be the gross sectional area with deductions for holes as specified in 5.8.
- (4) The effective sectional area for parts in shear should be taken as follows, subject to (5) 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 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 should 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 should be computed from the whole area of the cross section, having regard to the distribution of flexural stresses.

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

Table 2. Effective length L for a cantilever under normal loading conditions without intermediate lateral restraint

Restraining Conditions		Effective Length
At support	At tip	
Built-in laterally and torsionally	Free	$0.8 \times \text{length}$
	Lateral restraint only. (at top flange)	$0.7 \times \text{length}$
	Torsional restraint	$0.6 \times \text{length}$
	Lateral and torsional restraint	$0.5 \times \text{length}$
Continuous, with lateral and torsional restraint	Free	$1.0 \times \text{length}$
	Lateral restraint only (at top flange)	$0.9 \times \text{length}$
	Torsional restraint only	$0.8 \times \text{length}$
	Lateral and torsional restraint	$0.7 \times \text{length}$
Continuous, with lateral restraint only	Free	$3.0 \times \text{length}$
	Lateral restraint only (at top flange)	$2.7 \times \text{length}$
	Torsional restraint only	$2.4 \times \text{length}$
	Lateral and torsional restraint	$2.1 \times \text{length}$

Note: The values in this table do not apply when moment is applied at the tip.

6.10.2 FLANGES

In bolted construction, flange angles should 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 should be kept to a minimum.

In exposed situations where flange plates are used, at least one plate of the top flange should 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 bolts should be provided, if necessary, to comply with the requirements of 10.2.7 and 10.2.8.

Each flange plate should be extended beyond its theoretical cut-off point, and the extension should contain sufficient bolts or weld 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 plate, should not exceed the amounts given in Table 3.

Table 3. Maximum outstand of flanges

Type of construction	Flange	Max. outstand
Plates with unstiffened edges	Compression	$\frac{250 t}{\sqrt{Y_s}}$
	Tension	20t
Channel sections or plates with continuously stiffened edges	Compression or tension	20t*

Where t is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates tacked together.

* 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, should be as required by 7.3.1.

6.10.3 FLANGE SPLICES

Flange joints should preferably not be located at points of maximum stress.

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

6.10.4 CONNECTION OF FLANGES TO WEB

The flanges of plate girders should be connected to the web by sufficient 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.

6.10.5 DISPERSION OF LOAD THROUGH FLANGE TO WEB

Where a load is applied direct to a flange, it should 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.

6.10.6 WEB PLATES

(1) Minimum thickness

The thickness t of the web plate should be not less than the following:

(a) For unstiffened webs:

$$\frac{d_1 \sqrt{Y_s}}{1340}$$

where d_1 is the clear distance between flange angles or, where there are no flange angles, the clear distance between flanges, ignoring fillets.

(b) For vertically stiffened webs:

1/180 of the smaller clear panel dimension and

$$\frac{d_2 \sqrt{Y_s}}{3200}$$

- (c) 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

$$\frac{d_1 \sqrt{Y_s}}{4000}$$

- (d) Where there is also a horizontal stiffener at the neutral axis of the girder:

1/180 of the smaller dimension in each panel and

$$\frac{d_1 \sqrt{Y_s}}{6400}$$

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 should the greater clear dimension of a web panel exceed 270t.

(2) Welded construction

The gap between web plates and flange plates should be kept to a minimum, and for fillet welds should not exceed 1 mm at any point before welding.

6.10.7 SPLICES IN WEBS

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

6.10.8 SIDE PLATES

Where additional plates are provided to augment the strength of the web, they should be placed on each side of the web and should be equal in thickness. The proportion of shear force which these reinforcing plates shall be deemed to resist should 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 should be carried beyond the theoretical cut off.

6.11 WEB STIFFENERS

Web stiffeners should be provided as follows:

(1) Load bearing stiffeners:

- (a) *Rolled I-beams and channels.* For rolled I-beams and channels, load bearing stiffeners should be provided at points of concentrated load (including points of support) where the concentrated load or reaction exceeds the value of:

$$p_c tB$$

Where p_c is the axial stress for struts as given in 7.1 for a slenderness ratio of:

$$\frac{d_3}{t} \sqrt{3}$$

Where t is the web thickness,

d_3 is the clear depth of web between root fillets,

B is 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 should 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 5.5.4).

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

- The flange through which the load (or reaction) is applied is effectively restrained against lateral movement relative to the other flange.
- Rotation of the loaded flange relative to the web is prevented.

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

- (b) *Plate girders.* For plate girders load bearing stiffeners should be provided at points of concentrated load and where the web would otherwise be overstressed (refer 5.5.4), and at points of support. Load bearing stiffeners should be symmetrical about the web where possible.
- (c) *All beams and girders.* Load bearing stiffeners should 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 should be taken about the axis parallel to the web of the beam or girder, and the working stress should 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 should 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 given in 6.6.

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

The ends of load bearing stiffeners should 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 should apply at both flanges.

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

For plate girders, where load bearing stiffeners at supports are the sole means of providing restraint against torsion (refer 6.9.1) the moment of inertia I of the stiffener should be not less than:

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

Where I is the moment of inertia of the stiffener about the centre line of the web plate (mm^4),

D is the overall depth of the girder in mm,

T is the maximum thickness of compression flange in mm,

R is the reaction on the bearing in kN;

W is the total load on the girder in kN.

In addition, the bases of the stiffeners in conjunction with the bearing of the girder should be capable of resisting a moment due to the horizontal force specified in the Note in 6.9.1.

(2) Intermediate stiffeners for plate girders

To limit web buckling, intermediate stiffeners should be provided as follows:

- (a) *Vertical stiffeners.* Vertical stiffeners should be provided throughout the length of the girder at a distance apart not greater than $1.5d_1$ (refer also 6.7.2) when the thickness of the web is less than $\frac{d_1 Y_s}{1340}$ where d_1 is defined as the clear distance between flange angles or, where there are no flange angles, the clear distance between flanges, ignoring fillets.

These stiffeners should be designed so that I is not less than:

$$1.5 \left(\frac{d_1^3 t^3}{S^2} \right)$$

Where I is the moment of inertia in mm^4 of the complete stiffeners about the centre of the web,

S is the maximum permitted clear distance in mm between stiffeners for thickness t ,

t is the minimum required thickness of web in mm;

d_1 is 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 Code of Practice (see above and 6.7.2), 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 should extend from flange to flange, but need not have the ends fitted to provide a tight bearing on the flange.

- (b) *Horizontal stiffeners.* Where horizontal stiffeners are used in addition to vertical stiffeners they should be as follows:

One horizontal stiffener (single or double) should 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:

$$\frac{d_2 \sqrt{Y_s}}{3200}$$

This stiffener should have a moment of inertia I not less than $4S_1 t^3$ 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 6.10.6.

A second horizontal stiffener (single or double) should be placed on the neutral axis of the girder when the thickness of the web is less than:

$$\frac{d_2 \sqrt{Y_s}}{4000}$$

This stiffener should have a moment of inertia I not less than $d_2 t^3$, where I and t are as defined in (i) above and d_2 also in mm.

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

- (c) *Outstand of stiffeners.* Unless the outer edge of each stiffener is continuously stiffened, the outstand of all stiffeners from the web should be not more than:

$$\text{For sections } \frac{250t}{\sqrt{Y_s}}$$

$$\text{for flats } 12t,$$

where t is the thickness of the section or flat.

- (d) *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 (a) above should be increased as shown below:

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

$$\text{Increase of } I = \frac{150 MD^2}{Et} \times 10^4 \text{ mm}^4$$

Lateral loading on stiffener:

$$\text{Increase of } I = \frac{0.3 PD^3}{Et} \times 10^4 \text{ mm}^4$$

Where M is the applied bending moment in kNm,

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

D is the overall depth of girder, in mm;

t is the thickness of web, in mm;

E is the Young's modulus in MPa.

- (e) *Connections of intermediate stiffeners to web.* Intermediate vertical and horizontal stiffeners not subjected to external loads should be welded to the web 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 should be added to the above value.

(3) Stiffeners for tubes

Where a tubular steel beam rests on an abutment or foundation it should 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 should be given to the local stresses set up and the method of application and stiffening should be such as to prevent the local stress from being excessive.

7. DESIGN OF COMPRESSION MEMBERS

7.1 AXIAL STRESSES IN STRUTS

7.1.1 UNCASED STRUTS

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

$$K_2 p_c = \frac{Y_s + (n + 1) C_o}{2} - \sqrt{\left[\frac{Y_s + (n + 1) C_o}{2} \right]^2 - Y_s C_o}$$

Where p_c is the permissible average stress in MPa,

K_2 is load factor or coefficient, taken as 1.7 for the purposes of this standard,

Y_s is the minimum yield stress in MPa,

C_o is the Euler critical stress $= \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2 200000}{(L/r)^2}$ MPa,

$n = 0.3(L/100r)^2$;

L/r = slenderness ratio is the effective length/radius of gyration.

Note: For values of L/r less than 30 the value of p_c should not exceed that obtained from linear interpolation between the value of p_c for $L/r = 30$ as found above, and a value of $p_c = Y_s/K_2$ for $L/r = 0$.

7.1.2 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 requirement of 7.6 and 7.7 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 not lower than grade 20.
- (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 either
 - (a) hard drawn wire conforming with BS 4482, or
 - (b) steel fabric conforming with BS 4483, or
 - (c) steel bars conforming with BS 4449.

The wire should be at least 5 mm diameter and the reinforcement should be in the form of links 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 not fewer than 4 longitudinal spacing bars.

- (e) *Connections of intermediate stiffeners to web.* Intermediate vertical and horizontal stiffeners not subjected to external loads should be welded to the web 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 should be added to the above value.

(3) Stiffeners for tubes

Where a tubular steel beam rests on an abutment or foundation it should 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 should be given to the local stresses set up and the method of application and stiffening should be such as to prevent the local stress from being excessive.

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

$$K_2 p_c = \frac{Y_s + (n + 1) C_o}{2} - \sqrt{\left[\frac{Y_s + (n + 1) C_o}{2} \right]^2 - Y_s C_o}$$

Where p_c is the permissible average stress in MPa,

K_2 is load factor or coefficient, taken as 1.7 for the purposes of this standard,

Y_s is the minimum yield stress in MPa,

C_o is the Euler critical stress $= \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2 200000}{(L/r)^2}$ MPa,

$n = 0.3(L/100r)^2$;

L/r = slenderness ratio is the effective length/radius of gyration.

Note: For values of L/r less than 30 the value of p_c should not exceed that obtained from linear interpolation between the value of p_c for $L/r = 30$ as found above, and a value of $p_c = Y_s/K_2$ for $L/r = 0$.

7.1.2 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 requirement of 7.6 and 7.7 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 not lower than grade 20.
- (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 either
 - (a) hard drawn wire conforming with BS 4482, or
 - (b) steel fabric conforming with BS 4483, or
 - (c) steel bars conforming with BS 4449.

The wire should be at least 5 mm diameter and the reinforcement should be in the form of links 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 not fewer than 4 longitudinal spacing bars.

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 should be taken as that of the uncased section.

In no case should the axial load on a cased strut exceed twice that which would be permitted on the uncased section, nor should 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 may be taken as assisting in carrying the load over its rectangular cross section. However any cover in excess of 75 mm from the overall dimensions of the steel section of the cased strut should be ignored. This cross section of concrete should 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 7.1.1) divided by 0.19 times the numerical value of p_{bc} for the grade of steel concerned.

Note: The above does not apply to steel struts of overall sectional dimensions greater than $1 \text{ m} \times 500 \text{ mm}$, the dimension of 1 m being measured parallel to the web, or to box sections.

7.1.3 ANGLES AS STRUTS:

- (1) For single-angle discontinuous struts connected to gussets or to a section either by bolting by not less than two 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 7.1.1 in which L is taken as 0.85 times the length of the strut, centre-to-centre of intersections at each end, and r is the minimum radius of gyration.

Single angle struts with single-bolted connections should be treated similarly, but the calculated stress should not exceed 80 per cent of the values given in 7.1.1, and the full length L centre-to-centre of intersections should be taken. In no case, however, should the ratio of slenderness for such single angle struts exceed 180.

- (2) For double angle discontinuous struts, back-to-back connected to both sides of a gusset or section by not less than two bolts in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length L should be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of restraint, and the calculated average stress should not exceed the values obtained from 7.1.1 for the ratio of slenderness based on the minimum radius of gyration about a rectangular axis of the strut. The angles should be connected together in their length so as to satisfy the requirements of 7.8
- (3) Double angle discontinuous struts back to back, connected to one side of a gusset or section by one or more bolts in each angle, or by the equivalent in welding, should be designed as for single angles in accordance with 7.1.3(1) and the angles should be connected together in their length so as to satisfy the requirements of 7.8
- (4) The provisions of 7.1.3 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 should be designed in accordance with 6.9 and 7.1.1

7.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

For the purpose of calculating L/r for compression members the effective length L should be taken as follows:

Effectively held in position and restrained in direction at both ends	$L = 0.7 \times \text{span}$
Effectively held in position at both ends and restrained in direction at one end	$L = 0.85 \times \text{span}$
Effectively held in position at both ends, but not restrained in direction	$L = \text{span}$
Effectively held in position and restrained in direction at one end, and at the other partially restrained in direction but not held in position	$L = 1.5 \times \text{span}$
Effectively held in position and restrained in direction at one end, but not held in position or restrained in direction at the other end	$L = 2.0 \times \text{span}$

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

7.3 DESIGN DETAILS

- 7.3.1 General. The thickness of an outstanding leg of any member in compression, unless the leg is stiffened, should be not less than:

$$\frac{\sqrt{Y_s}}{250} \text{ of the outstand}$$

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

$$\frac{1425}{\sqrt{Y_s}} t$$

Where t is the thickness of a single plate, or the total thickness of two or more plates effectively tacked together.

However where the width between any two adjacent line of connections or support exceeds the effective width b_e given by:

$$b_e = 45t \sqrt{\frac{240}{Y_s}} \frac{b/t - 13.5 \sqrt{240/Y_s}}{b/t - 6.0 \sqrt{240/Y_s}}$$

for sections built up by welding which are not stress relieved,

$$\text{or } b_e = 45t \sqrt{\frac{240}{Y_s}}$$

for other plates

where b is the unsupported width and t is the thickness, the effective section area and modulus should be computed using the effective width of all compression elements in place of their actual widths. The excess width should be omitted at the free edge of a projecting element and at the centre of a supported element. All other section properties, including the radius of gyration, should be based on the dimensions of the gross section.

7.3.2 JOINTS

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

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

Wherever possible, splices should 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 should be provided for.

7.4 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, should not exceed the following values:

For any member carrying loads resulting from dead loads with or without imposed loads and for single bolted single angle struts.

$$(L/r)_{\max} = 180$$

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. (See also 9.2)

$$(L/r)_{\max} = 250$$

7.5 ECCENTRICITY FOR STANCHIONS AND SOLID COLUMNS

- 7.5.1 For the purpose of determining the stress in a stanchion or column section, the beam reactions or similar loads should 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 should 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.
- 7.5.2. 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 should be divided in proportion to the moments of inertia of the stanchion sections, divided by their respective actual lengths.

7.6 LACING AND BATTENING

7.6.1 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 should not be varied throughout the length of the strut. The lacing of compression members should be proportioned to resist a total transverse shear force F_q at any point in the length of the member equal to 2.5 per cent of the axial force in the member, which shear force should be considered as divided equally among all transverse lacing systems in parallel planes.

Except for tie plates as specified below, lacing systems should not be combined with members or diaphragms perpendicular to the longitudinal axis of the strut unless all forces resulting from deformation are calculated and provided for in the lacing and its fastenings. Single lacing systems mutually opposed in direction on opposite sides of the main components should not be used unless the resulting torsional deformation is calculated and allowed for in the design.

For members carrying the bending stress calculated from the eccentricity of loading, applied end moments of lateral loading, the lacing should be provided to resist any shear due to bending, in addition to abovementioned 2.5 per cent.

7.6.2 DETERMINATION OF SECTION OF LACING BARS

The required section of lacing bars for compression members, or for tension members subject to bending, should be determined by using the appropriate permissible stresses, subject to the requirements in 7.6.4 below.

For tension members under direct stress only, the lacing bars should be subject to the requirements of 7.6.3 and 7.6.4 below.

The ratio L/r of the lacing bars for compression members should not exceed 140.

In welded construction these effective lengths should 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.

7.6.3 THICKNESS OF LACING BARS

The thickness of flat lacing bars should be not less than $1/40$ of the length between the inner end welds for single intersection lacing, and $1/60$ of this length for double intersection lacing welded at the intersections. Rolled section or tubes of equivalent strength may be used instead of flats.

7.6.4 ANGLE OF INCLINATION

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

7.6.5 SPACING

The maximum spacing of lacing bars, when connected by welding, should 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 lesser, where L is the distance between the centres of the connections of the lacing bars to each component.

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

7.6.6 ATTACHMENT TO MAIN MEMBERS

The welding of lacing bars to the main members should be sufficient to transmit the load in the bars. Where welded lacing bars overlap the main members, the amount of lap should 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 should 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 should be connected to each member by fillet welds on each side of the bar or by full penetration butt welds.

7.6.7 TIE PLATES

Laced compression members should be provided with tie plates at the ends of lacing systems and at points where the systems are interrupted. End tie plates should have a length of not less than the perpendicular distance between the centroids of the groups of welds connecting them to the main members, and intermediate tie plates should 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.

Where the tie plates are welded on, the welds should comply generally with the requirements in 7.7 for batten plates.

Tie plates and their fastenings should 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 should be not less than $1/50$ of the distance between the innermost connecting lines of welds, except where they are stiffened on their edges; where such stiffening is provided the plates should be not less than 8 mm thick.

- 7.6.8 As an alternative to the tie plates described in 7.6.7 above, a cross-braced panel of the same effective strength may be used.

7.7 BATTENED COMPRESSION MEMBERS

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

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

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

- 7.7.2 In battened compression members in which the ratio of slenderness about 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 should be such that the ratio of slenderness L/r of the lesser main component over that distance should 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 should 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.

- 7.7.3 The battens should be designed to carry the bending moments and shears arising from a transverse shear force F_q of 2.5 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.

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

Where d is the longitudinal distance centre-to-centre of battens,

a is the minimum transverse distance between the centroids of the welding,

F_q is the transverse shear force as defined in 7.7.3 above;

n is the number of parallel planes of battens.

- 7.7.5 End battens and those at points where the member is stayed in its length should have an effective length, longitudinally, of not less than the perpendicular distance between the centroids of the main members, and intermediate battens should have an effective length of not less than three-quarters of this distance, but in no case should the effective length of any batten be less than twice the width of one member in the plane of the battens.
- 7.7.6 The effective length of a batten should be taken as the longitudinal distance between end welds.
- 7.7.7 Battened plates should have a thickness of not less than 1/50 of the minimum distance between the innermost lines of connecting 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.
- 7.7.8 The length of weld connecting each longitudinal edge of the batten plate to a member should, in the aggregate, be not less than half the length of the batten plate, and at least one-third of the weld should be placed at each end of the longitudinal edge. In addition, the welding should 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 should 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.

- 7.7.9 Battened compression members composed of two angles forming a cruciform cross section should conform to the above requirements except as follows:
- (1) The battens should be in pairs placed one against the other, unless they are welded to form cruciform battens.
 - (2) A transverse shear force of $F_q/\sqrt{2}$ should be taken as occurring separately about each rectangular axis of the whole member.
 - (3) A longitudinal shear force of $F_l/\sqrt{2}$ and the moment $M/\sqrt{2}$ should 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 should be designed to resist a shear of 2.5 per cent of the total axial force.

F_q , F_l and M are as defined in clause 7.7.4 above, with $n = 1$.

- 7.7.10 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, should 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.
- 7.7.11 The slenderness ratio of a battened strut about the axis perpendicular to the battens should be calculated from:

$$\lambda = \sqrt{\lambda_m^2 + \lambda_c^2}$$

Where λ_m is the ratio L/r of the whole member about that axis;

λ_c is the maximum L/r of a main component (based on its minimum radius of gyration).

The effective length L of a main component should be taken as the clear distance between end welds or end fasteners of adjacent battens. The maximum slenderness of the strut as a whole should not be taken as less than $1.4 \lambda_c$.

7.8 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 should be connected together by 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 lesser.

In no case should the ends of the strut be connected together with less than two bolts or their equivalent in welding, and there should be not less than two additional connections spaced equidistant in the length of the strut. Where the members are separated back-to-back the bolts through these connections should 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 bolts should be used in each connection, one on the line of each gauge mark.

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

The bolts or welds in these connections should be sufficient to carry the shear forces and moments (if any) specified for battened struts, and in no case should the 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 bolting or welding should not be subjected to transverse loading in a plane perpendicular to the bolted or welded surfaces.

Where the components are in contact back-to-back, the spacing of the bolts or intermittent welds should not exceed the maximum spacing for compression members as given in clauses 10.2.6 and 11.2.7.

The slenderness ratio of the compound strut about the axis parallel to the connected surface should be calculated from:

$$\lambda = \sqrt{\lambda_m^2 + \lambda_c^2}$$

where λ_m is the ratio L/r of the whole member about that axis

λ_c is the maximum ratio L/r of a main component (based on its minimum radius of gyration)

The effective length L of a main component should be taken as the spacing centre to centre of the interconnections between the main components. The maximum slenderness of the strut as a whole should not be taken as less than $1.4 \lambda_c$.

7.9 STANCHION AND COLUMN BASES

7.9.1 GUSSETED BASES

For stanchions with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with the bearing area of the shaft should, 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 should be sufficient to transmit all the forces to which the base is subjected.

7.9.2 SLAB BASES

Stanchions with slab bases need not be provided with gussets, but fastenings should 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 should be:

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

Where t is the slab thickness in mm,

A is the greater projection of the plate beyond the stanchion in mm,

B is the lesser projection of the plate beyond the stanchion in mm,

w is the pressure or loading on the underside of the base in MPa;

P_{bct} is the permissible bending stress in the steel in MPa.

When the slab will not distribute the load uniformly or when the slab is not rectangular, calculations should be made to show that the specified stresses are not exceeded.

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 millimetre, of a square cap or base should be:

$$t = 10 \sqrt{\frac{90 W}{16 P_{bct}} \frac{D}{D - d}}$$

Where t is the thickness of the plate in mm,

W is the total axial loading in kN,

D is the length of the side of cap or base in mm,

d is the diameter of the reduced end, if any, of the column in mm;

P_{bct} is the permissible bending stress in the steel in MPa.

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 should be made based on the allowable stress of 185 MPa.

The cap or base plate should be not less than $1.5(d + 75)$ mm in length or diameter.

The area of the shoulder (the annular bearing area) should be sufficient to limit the stress in bearing, for the whole of the load communicated to the slab, to the maximum values given in 6.6 and resistance to any bending communicated to the shaft by the slab should 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 should 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 should be machined to give a perfect bearing and the welding should be sufficient to transmit the forces as required above for fastenings to slab bases.

7.10 *BASE PLATES AND BEARING PLATES*

The base plates and grillages of stanchions and the bearings and spreaders of beams and girders should 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 under the most adverse load and bending moments applicable to that foundation.

7.11 *GRILLAGE FOUNDATIONS*

Where grillage beams are enveloped in a solid block of dense concrete as specified in 6.5.1(2) the permissible working stresses given in this Code of Practice for uncased beams may be increased by $33 \frac{1}{3}$ per cent provided that:

- (1) The beams are spaced apart so that the distance between the edges of adjacent flanges is not less than 75 mm;
- (2) 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;
- (3) The concrete is properly compacted solid around all beams.
- (4) The concrete is not lower than grade 20.

These increased stresses shall not apply to hollow compound girders.

8. **DESIGN OF TENSION MEMBERS**

8.1 *AXIAL STRESSES IN TENSION*

The direct stress in axial tension p_t on the nett area of section should not exceed the value given by:

$$p_t = 0.60 Y_s$$

Where Y_s is the yield stress

8.2 *TENSILE STRESSES FOR ANGLES AND TEES*

8.2.1 *ECCENTRIC CONNECTIONS*

When eccentricity of loading occurs in connections of angles and tees in tension, the nett areas to be used in computing the mean tensile stress should be as given by the following rules:

- (1) Single angle connected through one leg. The nett sectional area A_e calculated from:

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

Where a_1 is the nett sectional area of the connected leg;

a_2 is the sectional area of the unconnected leg.

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

- (2) A pair of angles connected together along their length, as specified in 10.2.8 or 11.2.7 and attached to the same side of a gusset or the equivalent by only one leg of each angle. The nett sectional area A_e calculated from:

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

Where a_1 is the nett sectional area of the connected part;

a_2 is the sectional area of the unconnected part.

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

- (3) A single tee attached by its table only. As for (2) above.

8.2.2 DOUBLE ANGLES OR TEES PLACED BACK-TO-BACK

Double angles or tees 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 nett sectional area of the pair should be taken, provided the members are connected together along their length as given in 10.2.8 or 11.2.7.

Note: The area of the leg of an angle should 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 thickness by the depth minus the thickness of the table.

For lacing and battening of tension members refer 7.6.2.

9. CONSTRUCTIONAL DETAILS

9.1 BRACED FRAMES AND TRUSSES

Members of braced frames and trusses should, where practicable, be disposed symmetrically about the resultant line of force, and the connections should, where practicable, be arranged so that their centroid lies on the resultant of the forces they are intended to resist (see 9.6.3).

In the case of bolted, or welded trusses and braced frames, the strut members act under complex conditions and the effective length L should be taken as between 0.7 and 1.0 times the distance between centres of intersections, depending on the degree of end restraint (but see also 7.1.3).

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

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

9.2 ROOF TRUSSES

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.

The windward roof trusses in multiple-bay buildings should be designed to resist the appropriate wind loads estimated as set out in the Code of Practice on Wind Effects, and the component members of the sheltered trusses, if the trusses are of the same height, span, rise and spacing as the windward truss, should 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 should 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 should be proportioned to resist these greater forces or reversed forces.

9.3 PURLINS

All purlins should be designed in accordance with the requirements for uncased beams (refer 6.1 and 6.2 and 6.8.1)

9.4 SIDE AND END SHEETING RAILS

Side and end sheeting rails should be designed for wind pressures and vertical loads, if any; and the requirements of 6.9 as regards lateral stability of beams does not apply.

- (2) A pair of angles connected together along their length, as specified in 10.2.8 or 11.2.7 and attached to the same side of a gusset or the equivalent by only one leg of each angle. The nett sectional area A_e calculated from:

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

Where a_1 is the nett sectional area of the connected part;

a_2 is the sectional area of the unconnected part.

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

- (3) A single tee attached by its table only. As for (2) above.

8.2.2 DOUBLE ANGLES OR TEES PLACED BACK-TO-BACK

Double angles or tees 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 nett sectional area of the pair should be taken, provided the members are connected together along their length as given in 10.2.8 or 11.2.7.

Note: The area of the leg of an angle should 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 thickness by the depth minus the thickness of the table.

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

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9.4 SIDE AND END SHEETING RAILS

Side and end sheeting rails should be designed for wind pressures and vertical loads, if any; and the requirements of 6.9 as regards lateral stability of beams does not apply.

9.5 STEEL CASTINGS

The use of steel castings should be limited to bearings, junctions and other similar parts and the working stresses should not exceed the working stresses given in this Code of Practice for grade 250 steel.

Steel castings should conform to BS 3100, Grade A, 430 MPa with a minimum yield stress of 230 MPa.

9.6 CONNECTIONS

9.6.1 CLOSE TOLERANCE BOLTS, HIGH STRENGTH FRICTION GRIP BOLTS, BLACK BOLTS AND WELDING

As much of the work of fabrication as is reasonably practicable should 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 close tolerance bolts, high strength friction grip bolts or welding should be used. In all other cases bolts in clearance holes may be used provided that due allowance is made for any slippage.

9.6.2 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 close tolerance bolts should be considered as acting together to share the load. In all other connections sufficient of one type of fastening should be provided to communicate the entire force.

9.6.3 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 should 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 should provide adequate resistance to the induced bending moments.

Where the design is based on non-intersecting members at a joint, all stresses arising from the eccentricity of the members should be calculated and the stresses kept within the limits given in this Code of Practice.

9.6.4 PACKINGS

(1) Bolts through packings.

The number of bolts carrying shear through packing should be increased above the number required by normal calculations by 1.25 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 bolts required should be determined from the thickness of the thicker packing.

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

(2) Packing in welded construction.

Where a packing is used between two parts, the packing and the welds connecting it to each part should 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 should be transmitted through the welds alone, the welds being increased in size by an amount equal to the thickness of the packing.

9.7 LUG ANGLES

Lug angles connecting a channel-shaped member should, 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 should 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 should 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 should 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 should be capable of developing 20 per cent in excess of that force.

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

The effective connection of the lug angle should, as far as possible, terminate at the end of the member connected, and the fastening of the lug angle to the member should 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 should be taken as effective, notwithstanding the requirements of 8.2

10. BOLTING

10.1 ALLOWABLE STRESSES IN BOLTS

10.1.1 CALCULATION OF STRESSES

In calculating shear and bearing stresses the effective diameter of a bolt should be taken as its nominal diameter. In calculating the axial tensile stress in a bolt or a screwed tension rod the nett area should be used.

10.1.2 STRESSES IN BOLTS

The calculated stress in a bolt of strength grade 4.6 should not exceed the value given in Table 6.

The allowable calculated stress in a bolt (other than a high strength friction grip bolt) of higher grade than 4.6, should be that given in Table 6 multiplied by the ratio of its yield stress (or its stress at the permanent set limit of 0.2 per cent) or 0.7 times its tensile strength, whichever is the lesser, to 235 MPa.

Table 6. Allowable stresses in bolts (MPa)

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

10.1.3 BEARING STRESSES ON CONNECTED PARTS

The calculated bearing stress of a bolt on the parts connected by it should not exceed the value given in Table 7.

Where the end distance of a 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 bolt, the allowable bearing stress of that bolt on the connected part should be reduced in the ratio of the actual end distance to that limit.

Table 7. Allowance bearing stresses on connected parts (MPa)

Description of fasteners	Material of connected part		
	Grade 250	Grade 350	Grade 450
Close tolerance and turned bolts	300	420	480
Bolts in clearance holes	250	350	400

10.1.4 COMBINED SHEAR AND TENSION

Bolts subject to both shear and axial tension should be so proportioned that the calculated shear and axial stresses f_q and f_t , calculated in accordance with 10.1.1, do not exceed the respective allowable stresses p_q and p_t and that the quantity $f_q/p_q + f_t/p_t$ does not exceed 1.4.

10.1.5 HIGH STRENGTH FRICTION GRIP BOLTS

The above sections, 10.1.1 to 10.1.4 do not apply to high strength friction grip bolts.

10.2 BOLTS AND BOLTING:

10.2.1 BLACK BOLTS (black all over)

The dimensions should conform to those given for black bolts in BS 4190, 'ISO metric black hexagon bolts, screws and nuts.'

10.2.2 CLOSE TOLERANCE BOLTS

The dimensions should 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 required to develop the bearing load on the bolt.

10.2.3 TURNED BARREL BOLTS

The nominal diameter of the barrel should be in multiples of 2 mm and should be at least 2 mm larger in diameter than the screwed portion.

10.2.4 LOCKING OF NUTS

Wherever there is a risk of nuts becoming loose due to vibration or alternation of stresses, they should be securely locked.

10.2.5 MINIMUM PITCH

The distance between centres of bolts should be not less than 2.5 times the nominal diameter of the bolt.

10.2.6 MAXIMUM PITCH

The distance between centres of any two adjacent bolts (including tacking bolts) connecting together elements of compression or tension members should not exceed $32t$ or 300 mm where t is the thickness of the thinner outside plate.

The distance between centres of two adjacent bolts, in a line lying in the direction of stress, should not exceed $16t$ or 200 mm in tension members, and $12t$ or 200 mm in compression members. In the case of compression members in which forces are transferred through butting faces this distance should not exceed 4.5 times the diameter of the bolt for a distance from the abutting faces equal to 1.5 times the width of the member.

The distance between centres of any two consecutive bolts in a line adjacent and parallel to an edge of an outside plate should not exceed $100\text{ mm} + 4t$, or 200 mm in compression or tension members.

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

10.2.7 EDGE DISTANCE

The minimum distance from the centre of any hole to the edge of a plate should be in accordance with Table 8.

Table 8. 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

Where two or more parts are connected together a line of bolts should be provided at a distance of not more than $40 \text{ mm} + 4t$ 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 $12t$.

10.2.8 TACKING BOLTS

Where tacking bolts are necessary to satisfy the requirements above, such tacking bolts, not subject to calculated stress, should 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 should not exceed 16 times the thickness of the outside plate or 200 mm whichever is the less. In both cases, the lines of bolts should not be a greater distance apart than these pitches.

The foregoing requirements apply to struts and compression members generally, subject to the stipulations in this Code of Practice 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 bolts, with solid distance pieces where the parts are separated, should be provided at a pitch in line not exceeding 1 m.

11. WELDS AND WELDING

11.1 ALLOWABLE STRESSES IN WELDS

11.1.1 GENERAL

When electrodes complying with Sections 1 and 2 of BS 639 are used for the welding of grade 250 steel, or with Sections 1 and 4 of BS 639 are used for the welding of grade 350 steel, or with Sections 1 and 4 of BS 639 are used for the welding of grade 450 steel and the yield stress of an all-weld tensile test specimen is not less than 430 MPa when tested in accordance with Appendix D of BS 639, the following should apply:

- (1) Butt welds. Butt welds should be treated as parent metal with a thickness equal to the throat thickness (or a reduced throat thickness as given in 11.2 for certain butt welds) and the stresses should not exceed those allowed for the parent metal.
- (2) Fillet welds. The allowable stress in fillet welds, based on a thickness equal to the throat thickness, should be 115 MPa for grade 250 steel or 160 MPa for grade 350 steel or 195 MPa for Grade 450 steel.
- (3) 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 should apply.
- (4) When a weld is subject to a combination of stresses, the stresses should be combined as required in 5.5.3 and 5.5.4, the value of the equivalent stress f_e being not greater than that permitted for the parent metal.

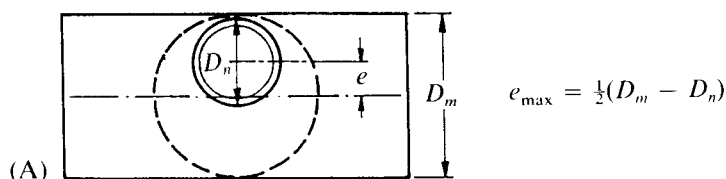
11.1.2 STEEL TUBES

- (1) A weld connecting two tubes end to end should be a butt weld.
- (2) 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° , should be either:

Type A: a butt weld throughout;

or Type B: a fillet weld throughout;

or Type 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.



$$e_{\max} = \frac{1}{2}(D_m - D_n)$$

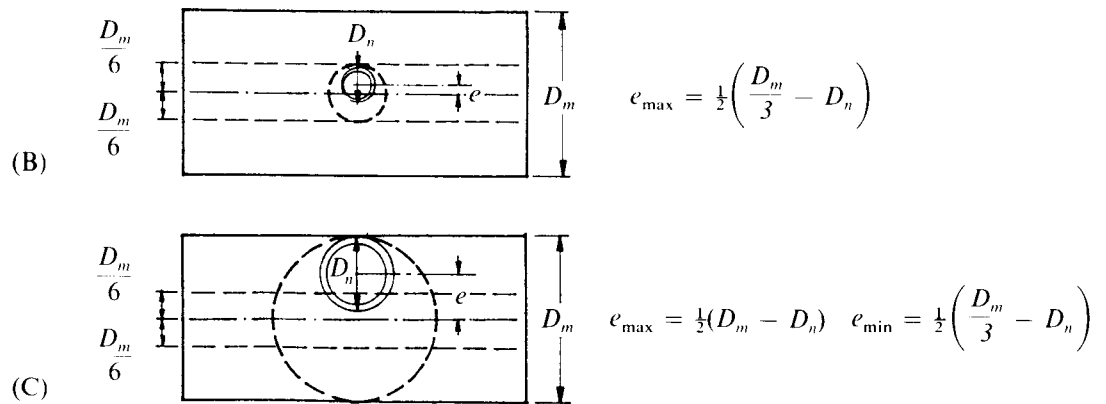


Fig. 11.2

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 should be used where the diameter of the branch tube is less than one-third of the diameter of the main tube, and Type C should 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 should 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) should be calculated on an area equal to the throat thickness at the throat multiplied by the length of the weld. (For a method of calculating the length of the weld, see Appendix D.)

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

- (3) 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° may be permitted only if adequate efficiency of the junction has been demonstrated.
- (4) A weld connecting the end of one tube to the surface of another, where the axes of the two tubes do not intersect, is subject to the provisions of 11.1.2(2) and 11.1.2(3) 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 (Fig. 11.2(A), (B) and (C).)
- (5) Where the end of the branch tube is flattened to an elliptical or other appropriate shape, (2), (3) and (4) above should apply, and for the application of (2) and (4) the diameter of the flattened portion of the tube should be measured in a plane perpendicular to the axis of the main tube.

11.2 DESIGN OF WELDS

The design of welds should be in accordance with the requirements of BS 5135, subject to the following conditions; and, where fatigue conditions apply, to the stipulation contained in the *Note* of 4.2.

11.2.1 INTERMITTENT BUTT WELDS

Intermittent butt welds should only be used to resist shear, and the effective length of any such weld should 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.

11.2.2 THROAT THICKNESS OF INCOMPLETE PENETRATION BUTT WELDS

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, should have a throat thickness of at least seven-eighths of the thickness of the thinner part joined. Evidence should 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, should be taken.

11.2.3 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, should 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 should in no case exceed 300 mm. This requirement should not be taken into account in complying with the requirements of 11.2.7 below and 7.8

Intermittent fillet welds should not be used for plate girders or where they would result in the formation of rust pockets.

11.2.4 LAP JOINTS

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

11.2.5 END RETURNS

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

11.2.6 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.

11.2.7 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 should 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, should not exceed 1000 mm in line whether they are opposite or staggered in respect of the two pairs of edges.

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

11.3 WELD ACCEPTANCE LEVELS

11.3.1 GENERAL

The following acceptance levels should be the minimum standards used. Any request for acceptance of welds which do not meet these standards may be examined on an individual basis using a fracture mechanics approach.

11.3.2 INSPECTION OF WELDS

All welds should be visually inspected in accordance with BS 5289. Butt welds in main members should be subject to non-destructive testing by either radiography or ultrasonic examination. The extent of these tests should be set by the Engineer taking into account the required performance of the welds and the completed structure.

11.4 VISUAL INSPECTION OF FUSION WELDED JOINTS

A weld subject to visual inspection should be acceptable if:

- (1) The weld is free from cracks.
- (2) The weld exhibits full fusion between parent and weld metal.
- (3) All craters are filled.
- (4) Welds exhibit the required size and profile.
- (5) Welds are free from excessive undercut and/or overlap.

The frequency of piping porosity in fillet welds and partial penetration corner butt welds should not exceed one in each 100 mm of length and the maximum pore diameter should not exceed 2.5 mm. Except for a fillet weld connecting stiffeners to web the sum of the diameters of piping porosity should not exceed 10 mm in any 25 mm and should not exceed 19 mm in any 300 mm of weld.

11.5 RADIOGRAPHY

Radiographic technique should be in accordance with BS 2600: Parts 1 and 2

Welds subject to radiographic inspection should be free from cracks and lack of fusion defects.

The total area of porosity as determined from a radiographic film should not exceed 3% of the projected area subject to a maximum acceptable pore diameter of 5 mm or $T/4$ whichever is the lesser, where T is the thickness of the metal.

Individual slag inclusions may be acceptable if:

- (1) The length does not exceed 10 mm in plate up to 20 mm thick.
- (2) They are acceptable when judged in accordance with Table 19 for thicker plate.
- (3) The aggregate length of a string of inclusions does not exceed 60 mm in any 150 mm length of weld.

11.6 ULTRASONIC EXAMINATION

The calibration, sensitivity and scanning technique employed should be in accordance with BS 3923:Part 1 or Part 2 as appropriate.

11.7 EVALUATION OF INDICATIONS

Indications which produce a response less than the primary reference level should be acceptable.

Indications which produce a response greater than the primary reference level should be investigated to determine their size.

- (1) Defects which cannot be sized as having a width greater than 2.5 mm may be regarded as point reflectors and should be acceptable provided that:
 - (a) Defect length does not exceed 10 mm for plate thickness up to 20 mm, 30 mm for plate thickness greater than 20 mm.
 - (b) The aggregate length of a string of inclusions does not exceed 60 mm in any 150 mm of weld.
- (2) Indications which produce a response greater than the reference level and that can be sized as having a width greater than 2.5 mm should be acceptable provided that:
 - (a) Defect length does not exceed 6 mm for plate up to 20 mm thick.
 - (b) Defects in thicker plate are judged acceptable in accordance with Table 9.
 - (c) The aggregate length of a string of inclusions does not exceeding 60 mm in any 150 mm of weld.
 - (d) Indications which produce a response which has a finite width greater than 2.5 mm and is interpreted as a crack or lack of fusion is unacceptable regardless of length.

11.8 TESTING OF CRANE GANTRY GIRDERS

100% of the welds joining the web to top flange of a crane gantry girder should be inspected in accordance with BS 3923:Part 1 or Part 2 as appropriate, but are subject to the following defect acceptance standard.

Indications which produce a response less than the primary reference level should be acceptable regardless of length.

Indications which produce a response greater than the primary reference level should be acceptable provided that:

- (1) The measured width of such defects does not exceed 15% of the web thickness or 2.5 mm whichever is the smaller.
- (2) The length of defects ascertained in accordance with (1) does not exceed 150 mm in any 600 mm length of weld or 5% of the length of the girder.

Welds on crane gantry girders will be marked with indelible crayon or other means to indicate that they have been inspected and found to be satisfactory.

Note: In the case of any dispute arising from the use of radiographic or ultrasonic testing, the ultrasonic examination and defect acceptance criteria should rule.

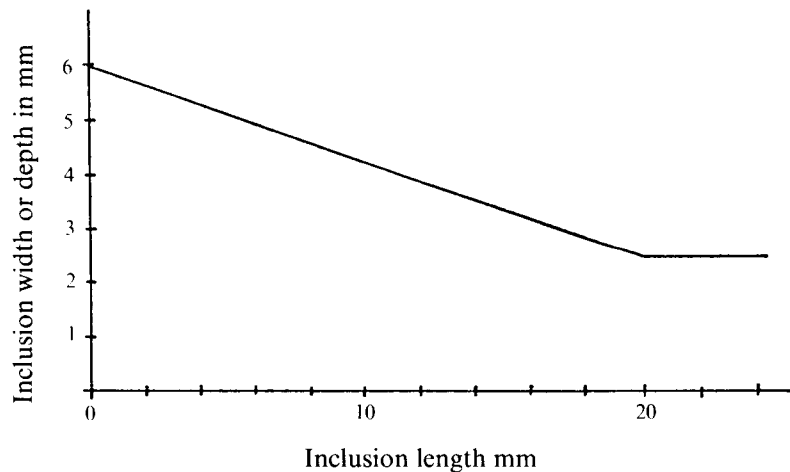
11.9 MAGNETIC PARTICLE INSPECTION

Magnetic Particle Inspection of welds should be carried out in accordance with the requirements of BS 6072. Extreme care should be taken to avoid arcing between work piece and probe when an electrical source is used to produce the required magnetic field. Stray arc strikes which do occur on the surface of the material should be dressed by grinding and inspected using either magnetic particle or dye penetrant techniques. On Grade 350 steels and any material having a carbon equivalent greater than 0.40 the use of permanent magnet is preferred as is the substitution of dye penetrant testing.

11.10 DYE PENETRANT TESTING

Dye Penetrant Testing of welds should be in accordance with BS 6443.

Table 9. Acceptance criteria for defects as determined by non destructive testing.



12. TREATMENT OF STEELWORK

12.1 AGAINST CORROSION

All surfaces which require a protective coating applied should have a surface preparation to a standard compatible with the protective system to be adopted. The type of treatment should be specified by the Engineer and be to the satisfaction of the Building Authority, and its application should be in strict accordance with the manufacturers specification. Necessary equipment to measure acceptance levels of surface preparation and protective coatings should be kept at the contractors place of work to ensure that checks can be made.

12.2 AGAINST FIRE

Fire engineering principles may be used to demonstrate adequate stability of the structure during fire, bearing in mind the consequences of structural collapse and safety of life. Where a "sprayed on" fire protection is acceptable it should be applied in strict accordance with the manufacturers specification. Where the sprayed on treatment also provides sufficient anti-corrosion protection then no additional form of corrosion protection may be required, provided that the structural member is not outside the weather envelope.

APPENDIX A: LOADING TEST

The methods of testing structures of unconventional design referred to in 5.1.3 of this standard should be as follows:

1. Acceptance tests

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

- (1) *Stiffness test.* In this test the structure or member should be subjected, in addition to the dead load, to a test load equal to 1.5 times the imposed and wind load, and this loading should 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 should be repeated. The structure should 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 should comply with 5.6
- (2) *Strength test.* The structure should be subjected, in addition to the dead load, to a test load equal to the sum of the dead load and twice the imposed and wind load, and this load should be maintained for 24 hours.

In the case of wind load, a load corresponding to twice the wind load should 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 should 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, should be fully tested, as described in (i) and (ii), supervised by the Engineer, but in addition, during the first application of the test load, particular note should be taken of the strain or deflection when the test load, at 1.5 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 should be subjected, in addition to the dead load, to a imposed test load, equal to 1.5 times the specified imposed load, in a manner and to an extent prescribed by the Engineer carrying out the test. This load should be maintained for 24 hours, during which time the maximum deflection should be noted. The check test should 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, should be considered satisfactory for occupancy after it has been returned to its original condition 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 bases.

Loading

- (1) *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.
- (2) *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.
- (3) *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 load 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: DEFLECTION

B1. General

The deformations of a structure and of its component members should be appropriate to the location, loading and function of the structure and the component members.

Deformations should be estimated using methods of analysis based on assumptions which reflect with reasonable accuracy the actual response of the structure to load.

It is not considered practicable in this Code of Practice to give limiting values for every deflection case, as each must be considered on merit. However, for guidance only, some values which have been found to give satisfactory service in areas not subject to earthquakes, are given below for certain structures and components.

B2. Deflection limits for specific cases

- (1) *Beams*. For a beam supporting a concrete floor intended for human accommodation and having a span not exceeding 9 m, the calculated deflection due to imposed load alone and based on the uncased section should not exceed—

$$\frac{\text{Span}}{360} \text{ or } \frac{\text{Cantilever Length}}{180}$$

- (2) *Purlins, Girts, Secondary Members*. For a purlin, or a girt, or a secondary member supporting metal sheeting and with a span not exceeding 6 m, the calculated deflection due to imposed load should not exceed—

$$\frac{\text{Span}}{180} \text{ or } \frac{\text{Cantilever Length}}{90}$$

- (3) *Industrial Buildings*. The horizontal deflection at eaves level of the internal frames of an industrial building, relative to the deflection in the same direction of the end wall at that level, should be appropriate to the capacity of the roof sheeting to accommodate the resulting shear distortion. Where no special provision is made for the sheeting to accommodate or resist this movement, the calculated eaves movement due to imposed and wind load should be limited to

$$\pm \frac{\text{Frame Spacing in End Bay}}{250}$$

- (4) *Tall Buildings*. The calculated total deflection due to imposed and wind loads for multistorey buildings should be limited to—

$$\frac{\text{Height of building}}{500}$$

The calculated inter-storey deflection should be limited to a value which will not cause unacceptable damage to internal or external finishes, and in no case should exceed

$$\frac{\text{storey height}}{400}$$

B3. Special conditions

In addition to the normal considerations which limit dead load deflection, both for aesthetic reasons and to prevent cracking of walls and other elements, consideration should be given to any special conditions which may apply. Some of these are:

- (1) *Moment Increase Due to Deflection*. There is a possibility that a structure may become unstable if a combination of high axial load and deflection in a transverse direction can give rise to high secondary bending moments. If the lateral deflection of a building is not restricted for other reasons this effect should be considered.
- (2) *Load Increase Due to Beam Deflection*. Overloading, increase in deflection, and possible instability may occur in beams as a consequence of the ponding of water on roofs of low slope.
- (3) *Damage to Cladding*. Relatively large lateral deflections may be acceptable in single-storey industrial structures since the stresses caused by secondary moments are usually small (see B3.1 above). However, such deflections may damage roof covering, particularly in the vicinity of stiff end walls. Similar damage can occur to walls and cladding.

- (4) *Resonance*. Imposed loads applied rhythmically to structure may cause resonance and hence cause deflections and stresses greater than would result if the maximum value of the load were applied statically. This may occur when a frequency component of the applied load is close to a natural frequency of the structure.

Imposed loads which are conducive to resonance include dancing, vehicular traffic, earthquake disturbance, unbalanced rotating machinery, and wind loads which result from the periodic shedding of vortices as can occur from tower-like structures and from roofs. In addition to general structural vibration, associated phenomena are 'springiness' in floors and the 'flutter' of roofs. A possible consequence of a repetitive load is metal fatigue.

- (5) *Deflection Analysis*. Calculated deflections may underestimate actual deflections where the method of analysis assumes a 'rigid' column zone at joints, and an adjustment, or calculation by another method (e.g. by assuming zero width columns) of the deflections may be necessary. Special attention should be given to this matter where the deflections are critical.