Indian Standard
CODE OF PRACTICE FOR GENERAL CONSTRUCTION IN STEEL
(*First Revision*)

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Indian Standard

CODE OF PRACTICE FOR GENERAL CONSTRUCTION IN STEEL

(Second Revision)

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Indian Standard

CODE OF PRACTICE FOR GENERAL CONSTRUCTION IN STEEL

(Second Revision)

0. FOREWORD

0.1 This Indian Standard (Second Revision) was adopted by the Indian Standards Institution on 25 April 1984, after the draft finalized by the Structural Engineering Sectional Committee had been approved by the Structural and Metal Division Council and the Civil Engineering Division Council.

0.2 The Steel Economy Programme was initiated by ISI in 1950's with the object of achieving economy in the use of structural steel by establishing rational, efficient and optimum standards for structural steel products and their use. IS : 800-1956 was the first in the series of Indian Standards brought out under this programme. The revision of this standard was taken up after the standard was in use for some time which was published in 1962 incorporating certain very important changes.

0.3 IS : 800 is a basic standard widely used and accepted by engineers, technical institutions, professional bodies and the industry. The committee while preparing the second revision has given careful consideration to the comments received on the standard during its usage. Consideration has also been given to the developments taking place in the country and abroad; necessary modifications and additions have therefore been incorporated to make the standard more useful.

0.4 In this revision the following major modifications have been effected:

a) Besides a general rearrangement of the clauses, formulae and the values have been given in SI units only.

b) Symbols used in this standard have been aligned to the extent possible with ISO 3898-1976 ' Basis for design of structures — Notation — General symbols ', and these have been listed in **1.3**.

c) All the Indian Standards referred to in this Code have been listed under **1.4**.

d) In view of the development and production of new varieties of medium and high tensile structural steels in the country, the scope of the Code has been modified permitting the use of any variety of structural steel provided the relevant provisions of the Code are satisfied.

e) Indian Standards are now available for rivets, bolts and other fasteners and reference has been made to these standards.

f) In view of the fact that the Code specifies a number of grades of steel with different yield strengths, the design parameter, the geometrical properties and permissible stresses have been expressed to the extent possible in terms of the yield strength of the material. Specific values have also been given for commonly used steels.

g) Recommendations regarding expansion joints have been added.

h) Keeping in view the developments in the design of steel structures there has been a general revision in the permissible stress values for steels and fasteners.

j) In IS : 800-1962, design by plastic theory had been permitted. In this revision detailed design rules have been included for design using plastic theory.

k) Specific provisions relating to limiting deflection have been added.

m) Effective length of columns has been dealt with in a greater detail. For normally encountered struts, a table has been given strictly on the basis of end conditions. The effective length of columns in framed structures and stepped columns in mill buildings have been specified on more exact basis.

n) The secant formula for axial compression has been dropped. In its place the Merchant Rankine formula has been specified with value of n, empirically fixed as 1·4.

p) *Bending stresses* — The method of calculating the critical stresses in bending compression f_{cb} has been simplified by expressing the formulae in terms of geometrical properties of the section. Merchant Rankine formula recommended for calculating permissible stresses in axial compression has been used for calculating permissible stresses in bending compression from the critical stresses, with value of n, empirically fixed as 1·4.

0.4.1 More rigorous analytical procedures than envisaged in this Code are available and can be made use of for finding effective lengths of compression members in determining elastic critical loads.

0.5 The original title of the code namely 'Code of practice for use of structural steel in general building construction' has now been modified as

'Code of practice for general construction in steel', since it was felt that the code is applicable to all types of steel structures and not limited to buildings only.

0.6 While preparing this Code, the practices prevailing in the field in the country have been kept in view. Assistance has also been derived from the following publications:

AS 1250-1981 SAA Steel structures code. Standards Association of Australia.

BS 449 (Part II)-1969 Specification for the use of structural steel in building; Part II Metric units. British Standards Institution.

AISC Specification for the design, fabrication and erection of structural steel for buildings. American Institute of Steel Construction.

SNIP-II-V3-72 Code of Practice for design of steel structures of the USSR State Committee for Construction.

SECTION 1 GENERAL

1.1 Scope

1.1.1 This code applies to general construction in steel. Specific provisions for bridges, chimneys, cranes, tanks, transmission line towers, storage structures, tubular structures and structures using cold formed light gauge sections, etc, are covered in separate codes.

1.1.2 The provisions of this code generally apply to riveted, bolted and welded constructions, using hot rolled steel sections.

1.1.3 This code gives only general guidance as regards the various loads to be considered in design . For actual loads to be used reference may be made to IS : 875-1964.

1.2 Terminology — For the purpose of this code the following definitions shall apply.

1.2.1 *Buckling Load* — The load at which a member or a structure as a whole collapses in service or buckles in a load test.

1.2.2 *Dead Loads* — The self weights of all permanent constructions and installations including the self weights of all walls, partitions, floors and roofs.

1.2.3 *Effective Lateral Restraint* — Restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a loaded strut, beam or girder from buckling to either side at the point of application of the restraint.

1.2.4 *Elastic Critical Moment* — The elastic moment which will initiate yielding or cause buckling.

1.2.5 *Factor of Safety* — The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

1.2.6 *Gauge* — The transverse spacing between parallel adjacent lines of fasteners.

1.2.7 *Imposed (Live) Load* — The load assumed to be produced by the intended use of occupancy including distributed, concentrated, impact and vibration and snow loads but excluding, wind and earthquake loads.

1.2.8 *Load Factor* — The numerical factor by which the working load is to be multiplied to obtain an appropriate design ultimate load.

1.2.9 *Main Member* — A structural member which is primarily responsible for carrying and distributing the applied load.

1.2.10 *Pitch* — The centre to centre distance between individual fasteners in a line of fastener.

1.2.11 *Secondary Member* — Secondary member is that which is provided for stability and or restraining the main members from buckling or similar modes of failure.

1.2.12 *Welding Terms* — Unless otherwise defined in this standard the welding terms used shall have the meaning given in IS : 812-1957.

1.2.13 *Yield Stress* — The minimum yield stress of the material in tension as specified in relevant Indian Standards.

1.3 Symbols — Symbols used in this Code shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this Code:

A	Cross-sectional area (A used with subscripts has been defined at appropriate place)
a, b	Respectively the greater and lesser projection of the plate beyond column
B	Length of side of cap or base
b_0	Width of steel flange in encased member
C_m	Coefficient

C	The distance centre to centre of battens
c	Distance between vertical stiffeners
c_1, c_2	Respectively the lesser and greater distances from the sections neutral axis to the extreme fibres
D	Overall depth of beam
d	Depth of girder — to be taken as the clear distance between flange angles or where there are no flange angles the clear distance between flanges ignoring fillets
d_0	Diameter of the reduced end of the column
d_1	i) For the web of a beam without horizontal stiffeners—the clear distance between the flanges, neglecting fillets or the clear distance between the inner toes of the flange angles as appropriate. ii) For the web of a beam with horizontal stiffeners — the clear distance between the horizontal stiffener and the tension flange, neglecting fillets or the inner toes of the tension flange angles as appropriate.
d_2	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
E	The modulus of elasticity for steel, taken as 2×10^5 MPa in this Code
f_y	Yield stress
f_{cb}	Elastic critical stress in bending
f_{cc}	Elastic critical stress in compression, also known as Euler critical stress.
g	Gauge
h	Outstand of the stiffener
I	Moment of inertia
K_b or K_c	Flexural stiffnesses
k_1, k_2	Coefficients
k	Distance from outer face of flange to web toe of fillet of member to be stiffened
L	Span/length of member
l	Effective length of the member
M	Bending moment
M_p	Maximum moment (plastic) capacity of a section
M_{pc}	Maximum moment (plastic) capacity of a section subjected to bending and axial loads

M_0	Lateral buckling strength in the absence of axial load
N	Number of parallel planes of battens
n	Coefficient in the Merchant Rankine formula, assumed as 1·4
P	Axial force, compressive or tensile
P_{ac}	Calculated maximum load capacity of a strut
P_{at}	Calculated maximum load capacity as a tension member
P_e	Euler load
P_y	Yield strength of axially loaded section
R	The reaction of the beam at the support
r	Radius of gyration of the section
S	Transverse distance between centroids of rivets groups or welding
s	Staggered pitch
T	Mean thickness of compression flange (T used with subscripts has been defined at appropriate place)
t	Thickness of web
V	Transverse shear
V_1	Longitudinal shear
V_y	Calculated maximum shear capacity of a section
W	Total load
w	Pressure or loading on the underside of the base
Z_p	Plastic modulus of the section
β	Ratio of smaller to larger moment
β_1, β_2	Stiffness ratio
λ	Slenderness ratio of the member; ratio of the effective length (l) to the appropriate radius of gyration (r)
λ_0	Characteristic slenderness ratio $= \sqrt{\dfrac{P_y}{P_e}}$
σ_{ac}	Maximum permissible compressive stress in an axially loaded strut not subjected to bending
σ_{at}	Maximum permissible tensile stress in an axially loaded tension member not subjected to bending
σ_{bs}	Maximum permissible bending stress in slab base
σ_{bc}	Maximum permissible compressive stress due to bending in a member not subjected to axial-force.
σ_{bt}	Maximum permissible tensile stress due to bending in a member not subjected to axial force

σ_{c} Maximum permissible stress in concrete in compression

σ_{e} Maximum permissible equivalent stress

σ_{p} Maximum permissible bearing stress in a member

σ_{pf} Maximum permissible bearing stress in a fastener

σ_{sc} Maximum permissible stress in steel in compression

σ_{tf} Maximum permissible stress in axial tension in fastener

$\sigma_{ac,\ cal.}$ Calculated average axial compressive stress

$\sigma_{at,\ cal.}$ Calculated average stress in a member due to an axial tensile force

$\sigma_{bc,\ cal.}$ Calculated compressive stress in a member due to bending about a principal axis

$\sigma_{bt,\ cal.}$ Calculated tensile stress in a member due to bending about both principal axes

τ_{va} Maximum permissible average shear stress in a member

τ_{vm} Maximum permissible shear stress in a member

τ_{vf} Maximum permissible shear stress in fastener

θ Ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge

ν Coefficient

Ψ Ratio of total area of both the flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment

ω Ratio of moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges each calculated about its own axis parallel to the *y-y* axis of the girder, at the point of maximum bending moment.

NOTE — The subscript *x*, *y* denote the *x-x* and *y-y* axes of the section respectively. For symmetrical sections, *x-x* denotes the major principal axis whilst *y-y* denotes the minor principal axis.

1.4 Reference to Other Standards — All the standards referred to in this Code are listed as under; and their latest version shall be applicable:

IS :

226-1975 Structural steel (standard quality) (*fifth revision*)

456-1978 Code of practice for plain and reinforced concrete (*third revision*)

696-1972 Code of practice for general engineering drawings (*second revision*)

IS :

786-1967 (Supplement) SI supplement to Indian Standard conversion factors and conversion tables (*first revision*)

812-1957 Glossary of terms relating to welding and cutting of metals

813-1961 Scheme of symbols for welding

814 Covered electrodes for metal arc welding of structural steels:

814 (Part 1)-1974 Part 1 For welding products other than sheets (*fourth revision*)

814 (Part 2)-1974 Part 2 For welding sheets (*fourth revision*)

816-1969 Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)

817-1966 Code of practice for training and testing of metal arc welders (*revised*)

819-1957 Code of practice for resistance spot welding for light assemblies in mild steel

875-1964 Code of practice for structural safety of buildings: Loading standards (*revised*)

919-1963 Recommendations for limits and fits for engineering (*revised*)

961-1975 Structural steel (high tensile) (*second revision*)

962-1967 Code of practice for architectural and building drawings (*first revision*)

1024-1979 Code of practice for use of welding in bridges and structures subject to dynamic loading (*first revision*)

1030-1982 Carbon steel castings for general engineering purposes (*second revision*)

1148-1973 Hot-rolled steel rivet bars (up to 40 mm diameter) for structural purposes (*second revision*)

1149-1982 High tensile steel rivet bars for structural purposes

1261-1959 Code of practice for seam welding in mild steel

1278-1972 Filler rods and wires for gas welding (*second revision*)

1323-1962 Code of practice for oxy-acetylene welding for structural work in mild steel (*revised*)

1363-1967 Black hexagon bolts, nuts and lock nuts (diameter 6 to 39 mm) and black hexagon screws (diameter 6 to 24 mm) (*first revision*)

1364-1967 Precision and semi-precision hexagon bolts, screws, nuts and lock nuts (diameter range 6 to 39 mm) (*first revision*)

IS :

1367-1967	Technical supply conditions for threaded fasteners (*first revision*)
1393-1961	Code of practice for training and testing of oxy-acetylene welders
1395-1971	Molybdenum and chromium molybdenum vanadium low alloy steel electrodes for metal arc welding (*third revision*))
1477	Code of practice for painting of ferrous metals in buildings:
1477 (Part 1)-1971	Part 1 Pretreatment (*first revision*)
1477 (Part 2)-1971	Part 2 Painting
1893-1975	Criteria for earthquake resistant design of structures (*third revision*)
1929-1961	Rivets for general purposes (12 to 48 mm diameter)
1977-1975	Structural steel (ordinary quarity) (*second revision*)
2062-1984	Weldable structural steel (*third revision*)
2155-1962	Rivets for general purposes (below 12 mm diameter)
3613-1974	Acceptance tests for wire-flux combinations for submerged-arc welding of structural steels (*first revision*)
3640-1967	Hexagon fit bolts
3757-1972	High-tensile friction grip bolts (*first revision*)
4000-1967	Code of practice for assembly of structural joints using high tensile friction grip fasteners
5369-1975	General requirements for plain washers and lock washers (*first revision*)
5370-1969	Plain washers with outside diameter 3 × inside diameter
5372-1975	Taper washers for channels (ISMC) (*first revision*)
5374-1975	Taper washers for I-beams (ISMB) (*first revision*)
6419-1971	Welding rods and bare electrodes for gas shielded arc welding of structural steel
6560-1972	Molybdenum and chromium-molybdenum low alloy steel welding rods and base electrodes for gas shielded arc welding
6610-1972	Heavy washers for steel structures
6623-1972	High tensile friction grip nuts
6639-1972	Hexagon bolts for steel structures
6649-1972	High tensile friction grip washers

7205-1973	Safety code for erection of structural steel work
7215-1974	Tolerances for fabrication of steel structures
7280-1974	Bare wire electrodes for submerged arc welding of structural steels
7307 (Part 1)-1974	Approval tests for welding procedures: Part 1 Fusion welding of steel
7310 (Part 1)-1974	Approval tests for welders working to approved welding procedures: Part 1 Fusion welding of steel
7318 (Part 1)-1974	Approval tests for welders when welding procedure is not required: Part 1 Fusion welding of steel
8500-1977	Weldable structural steel (medium and high strength qualities)
9595-1980	Recommendations for metal arc welding of carbon and carbon manganese steels

1.5 Units and Conversion Factors — The SI system of units is applicable to this code. For conversion of system of units to another system, IS : 786-1967 (supplement) may be referred.

1.6 Standard Dimensions, Form and Weight

1.6.1 The dimensions, form, weight, tolerances of all rolled shapes and other members used in any steel structure shall, wherever available conform to the appropriate Indian Standards.

1.6.2 The dimensions, form, weight, tolerances of all rivets, bolts, nuts, studs, etc, shall conform to the requirements of appropriate Indian Standards, wherever available.

1.7 Plans and Drawings

1.7.1 Plans, drawings and stress sheet shall be prepared according to IS : 696-1972 and IS : 962-1967.

1.7.1.1 *Plans* — The plans (design drawings) shall show the complete design with sizes, sections, and the relative locations of the various members. Floor levels, column centres, and offsets shall be dimensioned. Plans shall be drawn to a scale large enough to convey the information adequately. Plans shall indicate the type of construction to be employed; and shall be supplemented by such data on the assumed loads, shears, moments and axial forces to be resisted by all members and their connections, as may be required for the proper preparation of shop drawings. Any special precaution to be taken in the erection of structure from the design consideration, the same shall also be indicated in the drawing.

1.7.1.2 *Shop drawings* — Shop drawings, giving complete information necessary for the fabrication of the component parts of the structure including the location, type, size, length and detail of all welds, shall be prepared in advance of the actual fabrication. They shall clearly distinguish between shop and field rivets, bolts and welds. For additional information to be included on drawings for designs based on the use of welding, reference shall be made to appropriate Indian Standards. Shop drawings shall be made in conformity with IS : 696-1972 and IS : 962-1967. A marking diagram allotting distinct identification marks to each separate part of steel work shall be prepared. The diagram shall be sufficient to ensure convenient assembly and erection at site.

1.7.2 Symbols for welding used on plans and shop drawings shall be according to IS : 813-1961.

SECTION 2 MATERIALS

2.1 Structural Steel — All structural steels used in general construction coming under the purview of this code shall, before fabrication conform to IS : 226-1975, IS : 961-1975, IS : 1977-1975, IS : 2062-1984, and IS : 8500-1977 as appropriate.

2.1.1 Any structural steel other than those specified in **2.1** may also be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

2.2 Rivets — Rivets shall conform to IS : 1929-1961 and IS : 2155-1962 as appropriate.

2.2.1 *High Tensile Steel Rivets* — High tensile steel rivets, if used, shall be manufactured from steel conforming to IS : 1149-1982.

2.3 Welding Consumables

2.3.1 Covered electrodes shall conform to IS : 814 (Part 1)-1974, IS : 814 (Part 2)-1974 or IS : 1395-1971 as appropriate.

2.3.2 Filler rods and wires for gas welding shall conform to IS : 1278-1972.

2.3.3 The bare wire electrodes for submerged-arc welding shall conform to IS : 7280-1974. The combination of wire and flux shall satisfy the requirements of IS : 3613-1974.

2.3.4 Filler rods and bare electrodes for gas shielded metal arc welding shall conform to IS : 6419-1971 and IS : 6560-1972 as appropriate.

2.4 Steel Castings — Steel castings shall conform to grade 23-45 of IS : 1030-1982.

2.5 Bolts and Nuts — Bolts and nuts shall conform to IS : 1363-1967, IS : 1364-1967, IS : 1367-1967, IS : 3640-1967, IS : 3757-1972, IS : 6623-1972, and IS : 6639-1972 as appropriate.

2.6 Washers — Washers shall conform to IS : 5369-1975, IS : 5370-1969, IS : 5372-1975, IS : 5374-1975, IS : 6610-1972, and IS : 6649-1972 as appropriate.

2.7 Cement Concrete — Cement concrete used in association with structural steel shall comply with the appropriate provisions of IS : 456-1978.

2.8 Other Materials — Other materials used in association with structural steel work shall conform to appropriate Indian Standards.

SECTION 3 GENERAL DESIGN REQUIREMENTS

3.1 Types of Loads

3.1.1 For the purpose of computing the maximum stresses in any structure or member of a structure, the following loads and load effects shall be taken into account, where applicable:

a) Dead loads;
b) Imposed loads;
c) Wind loads;
d) Earthquake loads;
e) Erection loads; and
f) Secondary effects due to contraction or expansion resulting from temperature changes, shrinkage, creep in compression members, differential settlements of the structure as a whole and its components.

3.1.1.1 Dead loads, imposed loads and wind loads to be assumed in design shall be as specified in IS : 875-1964.

3.1.1.2 Imposed loads arising from equipment, such as cranes, and machines to be assumed in design shall be as per manufacturers/suppliers data (*see* **3.4.2.4**).

3.1.1.3 Earthquake loads shall be assumed as per IS : 1893-1975.

3.1.1.4 The erection loads and temperature effects shall be considered as specified in **3.2** and **3.3**.

3.2 Erection Loads

3.2.1 All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment including all loads due to operation of such equipment, shall be considered as ' erection loads '. Proper provision shall be made, including temporary bracings to take care of all stresses due to erection loads. The structure as a whole and all parts of the structure in conjuction with the temporary bracings shall be capable of sustaining these erection loads, without exceeding the permissible stresses as specified in this code subject to the allowable increase of stresses as indicated in **3.9**. Dead load, wind load and also such parts of the live load as would be imposed on the structure during the period of erection shall be taken as acting together with the erection loads.

3.3 Temperature Effects

3.3.1 Expansion and contraction due to changes in temperature of the materials of a structure shall be considered and adequate provision made for the effects produced.

3.3.2 The temperature range varies for different localities and under different diurnal and seasonal conditions. The absolute maximum and minimum temperatures which may be expected in different localities in the country are indicated on the maps of India in Appendices A and B, respectively. These appendices may be used for guidance in assessing the maximum variations of temperature for which provision for expansion and contraction has to be allowed in the structure.

3.3.3 The temperatures indicated on the maps in Appendices A and B are the air temperatures in the shade. The range of variation in temperature of the building materials may be appreciably greater or less than the variation of air temperature and is influenced by the condition of exposure and the rate at which the materials composing the structure absorb or radiate heat. This difference in temperature variations of the material and air should be given due consideration.

3.3.4 The co-efficient of expansion for steel shall be taken as 0·000 012 per degree centigrade per unit length.

3.4 Design Considerations

3.4.1 General — All parts of the steel framework of the structure shall be capable of sustaining the most adverse combination of the dead loads, prescribed imposed loads, wind loads, earthquake loads where applicable and any other forces or loads to which the building may reasonably be subjected without exceeding the permissible stresses specified in this standard.

3.4.2 *Load Combinations*

3.4.2.1 Load combinations for design purposes shall be the one that produces maximum forces and effects and consequently maximum stresses from the following combinations of loads:

a) Dead load + imposed loads,

b) Dead load + imposed loads + wind or earthquake loads, and

c) Dead load + wind or earthquake loads.

NOTE — In case of structures bearing crane loads, imposed loads shall include the crane effect as given in **3.4.2.4**.

3.4.2.2 Wind load and earthquake loads shall be assumed not to act simultaneously. The effect of both the forces shall be given separately.

3.4.2.3 The effect of cranes to be considered under imposed loads shall include the vertical loads, eccentricity effects induced by the vertical loads, impact factors, lateral (surge) and the longitudinal horizontal thrusts acting across and along the crane rail, respectively.

3.4.2.4 The crane loads to be considered shall be as indicated by the customer. In the absence of any specific indications the load combination shall be as follows:

a) Vertical loads with full impact from one loaded crane or two cranes in case of tandem operation together with vertical loads, without impact, from as many loaded cranes as may be positioned for maximum effect, alongwith maximum horizontal thrust (surge) from one crane only or two cranes in case of tandem operation;

b) For multibay multicrane gantries — loads as specified in (a) above, subject to consideration of cranes in maximum of any two bays of the building cross section;

c) The longitudinal thrust on a crane track rail shall be considered for a maximum of two loaded cranes on the track; and

d) Lateral thrust (surge) and the longitudinal thrust acting respectively across and along the crane rail shall not be assumed to act simultaneously. The effect of both the forces, shall, however, be investigated separately.

3.4.2.5 While investigating the effect of earthquake forces the resulting effect from dead loads of all cranes parked in each bay positioned for maximum effect shall be considered.

3.4.2.6 The crane runway girders supporting bumpers shall be checked for bumper impact loads.

3.4.2.7 Stresses developed due to secondary effects such as handling, erection, temperature effects, settlement of foundations shall be appropriately added to the stresses calculated from the combination of loads stated in **3.4.2.1**. The total stresses thus calculated shall be within the permissible limits as specified in **3.9**.

3.4.3 *Methods of Design* — The following methods may be employed for the design of the steel framework:

a) Simple design,

b) Semi-rigid design, and

c) Fully rigid design.

3.4.4 *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 consequence the structure may, for the purpose of design, be assumed to be pin-jointed.

3.4.4.1 The method of simple design involves the following assumptions:

a) Beams are simply supported;

b) All connections of beams, girders or trusses are virtually flexible and are proportioned for the reaction shears applied at the appropriate eccentricity;

c) Members in compression are subjected to forces applied at the appropriate eccentricities (*see* **5.3.3**) with the effective length given in **5.2**; and

d) Members in tension are subjected to longitudinal forces applied over the net area of the section, as specified under **3.6.2** and **4.2.1**.

3.4.5 *Semi-Rigid Design* — This method, as compared with the simple design method, permits a reduction in the maximum bending moment in beams suitably connected to their supports, so as to provide a degree of direction fixity, and in the case of triangulated frames, it permits account being taken of the rigidity of the connections and the moment of interaction of members. In cases where this method of design is employed, calculations based on general or particular experimental evidence shall be made to show that the stresses in any part of the structure are not in excess of those laid down in the code. Stress investigations may also be done on the finished structure for assurance that the actual stresses under specific design loads are not in excess of those laid down in the standard.

3.4.6 *Fully Rigid Design* — This method as compared to the methods of simple and semi-rigid designs gives the greatest rigidity and economy in the weight of steel used when applied in appropriate cases. The end connections of members of the frame shall have sufficient rigidity to hold the original angles between such members and the members they connect virtually unchanged. Unless otherwise specified, the design shall be based on theoretical methods of elastic analysis and the calculated stresses shall conform to the relevant provisions of this standard. Alternatively, it shall be based on the principles of plastic design as given in Section 9 of the code.

3.4.7 *Experimentally Based Design* — Where structure is of non-conventional or complex nature, the design may be based on full scale or model tests subject to the following conditions:

a) A full scale test of prototype structure may be done. The prototype shall be accurately measured before testing to determine the dimensional tolerance in all relevant parts of the structure; the tolerances then specified on the drawing shall be such that all successive structures shall be in practical conformity with the prototype. Where the design is based on failure loads, a load factor of not less than 2·0 on the loads or load combinations given in **3.4.2** shall be used. Loading devices shall be previously calibrated and care shall be exercised to ensure that no artificial restraints are applied to the prototype by the loading systems. The distribution and duration of forces applied in the test shall be representative of those to which the structure is deemed to be subjected.

b) In the case where design is based on the testing of a small scale model structure, the model shall be constructed with due regard for the principles of dimensional similarity. The thrusts, moments and deformations under working loads shall be determined by physical measurements made when the loadings are applied to simulate the conditions assumed in the design of the actual structure.

3.5 Geometrical Properties

3.5.1 *General* — The geometrical properties of the gross and the effective cross sections of a member or part thereof shall be calculated on the following basis:

a) The properties of the gross cross section shall be calculated from the specified size of the member or part thereof.

b) The properties of the effective cross section shall be calculated by deducting from the area of the gross cross section the following:

   i) The sectional area in excess of effective plate width, as given in **3.5.2**, and

   ii) The sectional areas of all holes in the section, except that for parts in compression (*see* **3.6**).

3.5.2 *Plate Thickness*

3.5.2.1 If the projection of a plate or flange beyond its connection to a web, or other line of support or the like, exceeds the relevant values given in (a), (b) and (c) below, the area of the excess flange shall be neglected when calculating the effective geometrical properties of the section.

a) Flanges and plates in compression with unstiffened edges	$\frac{256 T_1}{\sqrt{f_y}}$ subject to a maximum of $16 T_1$
b) Flanges and plates in compression with stiffened edges	$20 T_1$ to the innermost face of the stiffening
c) Flanges and plates in tension	$20 T_1$

NOTE 1 — Stiffened flanges shall include flanges composed of channels or I-sections or of plates with continuously stiffened edges.

NOTE 2 — 'T_1' denotes the thickness of the flange of a section or of a plate in compression, or the aggregate thickness of plates, if connected together in accordance with the provisions of Section 8, as appropriate.

NOTE 3 — The width of the outstand of members referred above shall be taken as follows:

Type	*Width of Outstand*
Plates	Distance from the free edge to the first row of rivets or welds
Angle, channels, Z-sections and stems of tee sections	Nominal width
Flange of beam and tee sections	Half the nominal width

3.5.2.2 Where a plate is connected to other parts of a built up member along lines generally parallel to the longitudinal axis of the member, the width between any two adjacent lines of connections or supports shall not exceed the following:

a) For plates in uniform compression — $\frac{1\,440 T_1}{\sqrt{f_y}}$ subject to a maximum of $90 T_1$

However, where the width exceeds —

$\frac{560 T_1}{\sqrt{f_y}}$, subject to a maximum of $35 T_1$ for welded plates which are not stressed relieved, or

$\frac{800 T_1}{\sqrt{f_y}}$, subject to a maximum of $50 T_1$ for other plates,

the excess width shall be assumed to be located centrally and its sectional area shall be neglected when calculating the effective geometrical properties of the section.

b) For plates in uniform tension — $100 T_1$. However where the width exceeds $60\ T_1$, the excess width shall be assumed to be located centrally and its sectional area shall be neglected when calculating the geometrical properties of the section.

In this rule, T_1 shall be taken to be the thickness of the plate, irrespective of whether the plate is a flange or a web of the member.

3.5.2.3 The provisions contained in **3.5.2.1** and **3.5.2.2** shall not be applicable to box girders (where width/depth is greater than 0·2). In such cases strength is not usually governed by lateral buckling. However, in such cases check should be exercised for local buckling and yield stress of material.

3.5.2.4 For only the diaphragm of the box girder, all the provisions pertaining to size, thickness, spacing etc. as given in **3.5.2.1** and **3.5.2.2** for plate girders shall be applicable.

3.6 Holes

3.6.1 *Diameter* — In calculating the area to be deducted for rivets, bolts or pins, the diameter of the hole shall be taken.

3.6.1.1 In making deduction for rivets less than or equal to 25 mm in diameter, the diameter of the hole shall be assumed to be 1·5 mm in excess of the nominal diameter of the rivet unless specified otherwise. If the diameter of the rivet is greater than 25 mm, the diameter of the hole shall be assumed to be 2·0 mm in excess of the nominal diameter of the rivet unless specified otherwise.

3.6.1.2 In making deduction for bolts, the diameter of the hole shall be assumed to be 1·5 mm in excess of the nominal diameter of the bolt, unless otherwise specified.

3.6.1.3 For counter sunk rivets or bolts the appropriate addition shall be made to the diameter of the hole.

3.6.2 *Deduction for Holes*

3.6.2.1 Except as required in **3.6.2.2** the areas to be deducted shall be the sum of the sectional area of the maximum number of holes in any cross section at right angles to the direction of stress in the member for:

a) all axially loaded tension members,

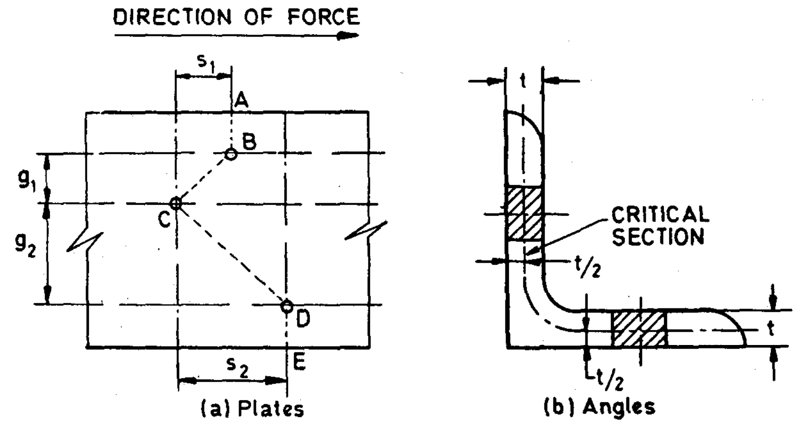
b) plate girders with d/t ratio exceeding the limits specified in **6.7.3.1**:

where

t = thickness of web, and

d = depth of the girder to be taken as the clear distance between flange angles or where there are no flange angles the clear distance between flanges ignoring fillets.

3.6.2.2 Where bolt or rivet holes are staggered, the area to be deducted shall be the sum of the sectional areas of all holes in a chain of lines extending progressively across the member, less $\frac{s^2t}{4g}$ for each line extending between holes at other than right angles to the direction of stress, where, s, g and t are respectively the staggered pitch, gauge, and thickness associated with the line under consideration [*see* Fig. 3.1(a)]. The chain of lines shall be chosen to produce the maximum such deduction. For non-planer sections, such as angles with holes in both legs, the gauge, g, shall be the distance along the centre of the thickness of the section between hole centres [*see* Fig. 3.1(b)].



DEDUCTION = (Sum of sectional areas of holes B, C and D) $- \left[\frac{s_1^2t}{4g_1} + \frac{s_2^2t}{4g_2} \right]$

FIG. 3.1 STAGGERED PITCH, s, AND GAUGE, g

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 members as a whole, the value of any rivets or bolts joining the parts between such chains of holes shall be taken into account in determining the strength of the member.

3.7 Maximum Slenderness Ratio

3.7.1 The maximum slenderness ratio $\lambda \left(= \frac{l}{r} \right)$ of a beam, strut or tension member given in Table 3.1 shall not be exceeded. In this 'l' is the effective length of the member (*see* **5.2**) and 'r' is appropriate radius of gyration based on the effective section as defined in **3.5.1**.

TABLE 3.1 MAXIMUM SLENDERNESS RATIOS

Sl No.	Member	Maximum Slenderness Ratio λ
(1)	(2)	(3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress due to loads other than wind or seismic forces occurs	180
iii)	A member subjected to compression forces resulting from wind/earthquake forces provided the deformation of such member does not adversely affect the stress in any part ofthe structure	250
iv)	Compression flange of a beam	300
v)	A member normally acting as a tie in a roof truss or a bracing system but subject to possible reverse of stress resulting from the action of wind or earthquake forces	350
vi)	Tension members (other than pretensioned members)	400

3.8 Corrosin Protection — Minimum Thickness of Metal

3.8.1 *General* — Except where the provisions of subsequent clauses in this section require thicker elements of members, the minimum thickness of metal for any structural element shall be, as specified under **3.8.2** to **3.8.4**.

3.8.2 *Steelwork Directly Exposed to Weather* — Where the steel is directly exposed to weather and is fully accessible for cleaning and repainting, the thickness shall be not less than 6 mm and where the steel is directly exposed to weather and is not accessible for cleaning and repainting, the thickness shall be not less than 8 mm. These provisions do not apply to the webs of Indian Standard rolled steel joists and channels or to packings.

3.8.3 *Steelwork not Directly Exposed to Weather*

3.8.3.1 The thickness of steel in main members not directly exposed to weather shall be not less than 6 mm.

3.8.3.2 The thickness of steel in secondary members not directly exposed to weather shall be not less than 4·5 mm.

3.8.4 *Rolled Steel Beams and Channels* — The controlling thickness as specified under **3.8.2** and **3.8.3** for rolled beams and channels shall be taken as the mean thickness of flange, regardless of the web thickness.

3.8.5 The requirements of thicknesses specified under **3.8.2** to **3.8.4** do not apply to special light structural work or to sealed box section or to steel work in which special provision against corrosion, such as use of special paints has been made or to steelwork exposed to highly corrosive industrial fumes or vapour or saline atmosphere. In such cases the minimum thickness of structural and secondary members shall be mutually settled between the customer and the designer.

3.9 Increase of Stresses

3.9.1 *General* — Except as specified in **3.9.2** to **3.9.4**, all parts of the structure shall be so proportioned that the working stresses shall not exceed the specified values.

3.9.2 *Increase in Permissible Stresses in Members Proportioned for Occasional Loadings*

3.9.2.1 *Wind or earthquake loads*

a) *Structural steel and steel castings* — When the effect of wind or earthquake load is taken into account, the permissible stresses specified may be exceeded by $33\frac{1}{3}$ percent.

b) *Rivets, bolts and tension rods* — When the effect of the wind or earthquake load is taken into account, the permissible stresses specified may be exceeded by 25 percent.

3.9.2.2 *Erection loads*

a) *Secondary effects—without wind or earthquake loads* — For constructions where secondary effects are considered without wind or earthquake loads, the permissible stresses on the member or its connections as specified may be exceeded by 25 percent.

b) *Secondary effects combined with wind or earthquake loads* — When secondary effects are considered together with wind or earthquake loads, the increase in the permissible stresses shall be as specified in **3.9.2.1**.

3.9.2.3 In no case shall a member or its connections have less carrying capacity than that needed if the wind or earthquake loads or secondary effects due to erection loads are neglected.

3.9.3 *Increase in Permissible Stresses for Design of Gantry Girders and Their Supporting Structures* — While considering the simultaneous effects of vertical and horizontal surge loads of cranes for the combination given in **3.4.2.3** and **3.4.2.4** the permissible stresses may be increased by 10 percent.

3.9.4 Where the wind load is the main load acting on the structure, no increase in the permissible stresses is allowed.

3.10 Fluctuation of Stresses

3.10.1 Members subjected to fluctuations of stresses are liable to suffer from fatigue failure caused by loads much lower than those which would be necessary to cause failure under a single application. The fatigue cracks are caused primarily due to stress concentrations introduced by constructional details. Discontinuities such as bolt or rivet holes, welds and other local or general changes in geometrical form cause such stress concentrations from which fatigue cracks may be initiated, and these cracks may subsequently propagate through the connected or fabricated members.

All details shall, therefore, be designed to avoid, as far as possible, stress concentrations likely to result in excessive reduction of the fatigue strength of members or connections. Care shall be taken to avoid sudden changes of shape of a member or part of a member, especially in regions of tensile stress or local secondary bending.

Except where specifically stated to the contrary, the permissible fatigue stresses for any particular detail are the same for all steels.

3.10.2 When subjected to fluctuations of stresses the permissible stresses shall be the basic stress stipulated in IS : 1024-1979 for different f_{min}/f_{max} and for different number of stress cycles and classes of constructional details.

The following provisions shall also be considered while determining the permissible stress in members subjected to fluctuations of stress:

a) While computing the value of f_{min}/f_{max} the effect of wind or earthquake temperature and secondary stresses shall be ignored

b) For plain steel in the as-rolled condition with no gas cut edges the constructional detail shall be considered as Class A of IS : 1024-1979.

c) For members of steel with yield stress 280 MPa and over, and fabricated or connected with bolts or rivets the construction details shall be considered as Class C of IS : 1024-1979.

   For members of steels with yield stress below 280 MPa, fabricated or connected with bolts or rivets the construction details shall be considered as Class D of IS : 1024-1979.

d) The value of f_{max} shall not exceed the permissible tensile or compressive fatigue stress as determined from IS : 1024-1979. Where co-existent bending and shear stresses are present, f_{max} shall be taken as the principal stress at the point under consideration.

3.11 Resistance to Horizontal Forces

3.11.1 In designing the steel framework of building, provisions shall be made by adequate moment connections or by a system of bracing to effectively transmit to the foundations all the horizontal forces, making due allowance for the stiffening effect of the walls and floors, where applicable.

3.11.2 When the walls, or walls and floors and/or roof are capable of effectively transmitting all of the horizontal forces directly to the foundations, the structural framework may be designed without considering the effect of wind.

3.11.3 Wind and earthquake forces are reversible and therefore calls for rigidity in both longitudinal and transverse directions. To provide for torsional effects of wind and earthquake forces bracings in plan should be provided and integrally connected with the longitudinal and transverse bracings to impart adequate torsional resistance to the structure.

3.11.3.1 In shed type buildings, adequate provisions shall be made by wind bracings to transfer the wind or earthquake loads from their points of action to the appropriate supporting members. Where the connections to the interior columns are so designed that the wind or earthquake loads are not transferred to the interior columns, the exterior columns shall be designed to resist the total wind or earthquake loads. Where the connections to the interior columns are so designed that the wind or earthquake effects are transferred to the interior columns also, both exterior and interior columns shall be designed on the assumption that the wind or earthquake load is divided among them in proportion to their relative stiffnesses. Columns also should be tested for proper anchorage to the trusses and other members to withstand the uplifting effect caused by excessive wind or earthquake pressure from below the roof.

3.11.3.2 Earthquake forces are proportional to the mass of structural component and the imposed load. Therefore earthquake forces should be applied at the centre of gravity of all such components of loads and their transfer to the foundation should be ensured (*see* IS : 1893-1975).

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

3.11.4 *Foundations* — The foundations of a building or other structure shall be so designed as to ensure such rigidity and strength as have been allowed for in the design of the superstructure, including resistance to all forces.

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

3.12 Stability

3.12.1 The stability of the structure as a whole or of any part of it shall be investigated, and weight or anchorage shall be provided so that the least restoring moment and anchorage, shall be not less than the sum of 1·2 times the maximum overturning moment due to dead load and 1·4 times the maximum overturning moment due to imposed loads and wind or earthquake loads.

3.12.1.1 In cases where dead load provides the restoring moment, only 0·9 times the dead load shall be considered. Restoring moment due to imposed loads shall be ignored.

3.12.1.2 To ensure stability at all times, account shall be taken of probable variations in dead load during construction, rapair or other temporary measures. The effect on the load from the deflected or deformed shape of the structure or of individual elements of the lateral load resisting systems, may be considered as required.

NOTE 1 — In complying with the requirements of **3.12.1**, it is necessary to ascertain that the resulting pressures and shear forces to be communicated by the foundations to the supporting soil would not cause failure.

NOTE 2 — All individual members of the structure which have been designed for their dead and imposed loads, wind or earthquake loads to the permissible stresses stipulated in this code shall be deemed to be adequately covered for this margin of stability.

3.13 Limiting Deflection

3.13.1 *Limiting Vertical Deflection*

3.13.1.1 The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads causing deflection.

3.13.1.2 The deflection of member shall not be such as to impair the strength or efficiency of the structure and lead to damage to finishings. Generally, the maximum deflection should not exceed 1/325 of the span, but this limit may be exceeded in cases where greater deflection would not impair the strength or efficiency of the structure or lead to damage to finishings.

3.13.1.3 In the case of crane runway girder the maximum vertical deflection under dead and imposed loads shall not exceed the following values:

a) Where manually operated cranes are operated and for similar loads	$\frac{L}{500}$
b) Where electric overhead travelling cranes operate, up to 50*t*	$\frac{L}{750}$
c) Where electric overhead travelling cranes operate, over 50*t*	$\frac{L}{1\,000}$
d) Other moving loads such as charging cars, etc	$\frac{L}{600}$

where,

L = span of crane runway girder.

3.13.2 *Limiting Horizontal Deflection*

3.13.2.1 At the caps of columns in single storey buildings, the horizontal deflection due to lateral forces should not ordinarily exceed 1/325 of the actual length '*l*' of the column. This limit may be exceeded in cases where greater deflection would not impair the strength and efficiency of the structure or lead to damage to finishing.

3.13.2.2 The horizontal deflection at column cap level of columns supporting crane runway girders in the building shall not exceed limits as may be specified by the purchaser.

3.14 Expansion Joints

3.14.1 In view of the large number of factors involved in deciding the location, spacing and nature of expansion joints, provisions of expansion joints should be left to the discretion of the designer.

3.14.2 Structures in which marked changes in plan dimensions take place abruptly shall be provided with expansion joints at the section where such changes occur. Expansion joints shall be so provided that the necessary movement occurs with a minimum resistance at the joint. The structure adjacent to the joint should preferably be supported on separate columns but not necessarily on separate foundation.

3.14.3 The details as to the length of a structure where expansion joints have to be provided may be determined after taking into conrideration various factors such as temperature, exposure to weather and structural design, etc. For the purpose of general guidance the following provisions have been recommended:

a) If one set of column longitudinal bracing is provided at the centre of the building or building section, the length of the building section may be restricted to 180 metres in case of covered buildings and 120 metres in case of open gantries (*see* Fig. 3.2).

b) If one set of column longitudinal bracing are provided near centre of the building/section, the maximum centre line distance between the two sets of bracing may be restricted to 48 metres for covered buildings (and 30 metres for open gantries) and the maximum distance between centre of the bracing to the nearest expansion joint/end of building or section may be restricted to 90 metres (60 metres in case of open gantries). The maximum length of the building section thus may be restricted to 228 metres for covered buildings [and 150 metres for open gantries (*see* Fig. 3.3)].

c) The maximum width of the covered building section should preferably be restricted to 150 metres beyond which suitable provisions for the expansion joints may be made.

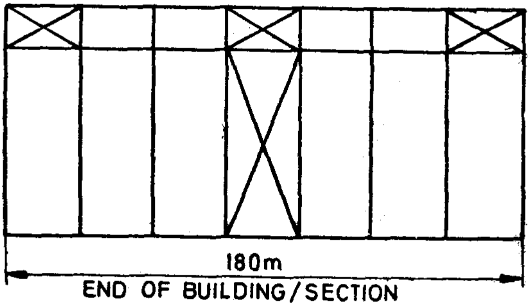


FIG. 3.2 MAXIMUM LENGTH OF BUILDING WITH ONE SET OF COLUMN BRACING

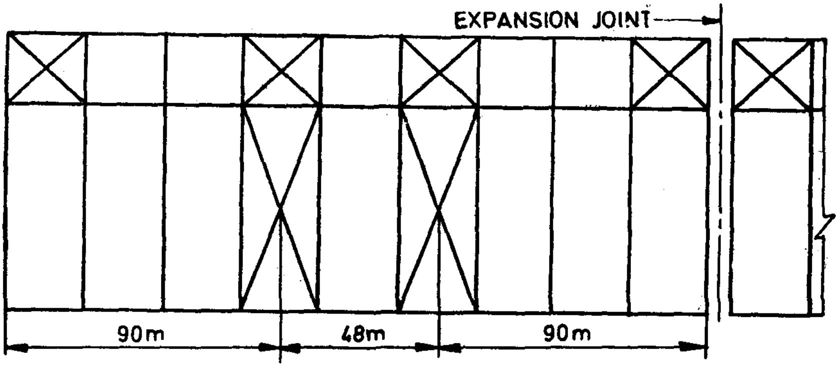


FIG. 3.3 MAXIMUM LENGTH OF BUILDINGS/SECTION WITH TWO SETS OF COLUMN BRACINGS

SECTION 4 DESIGN OF TENSION MEMBERS

4.1 Axial Stress

4.1.1 The permissible stress in axial tension, σ_{at}, in MPa on the net effective area of the sections shall not exceed:

$$\sigma_{at} = 0{\cdot}6 f_y$$

where,

f_y = minimum yield stress of steel, in MPa

4.2 Design Details

4.2.1 *Net Effective Areas for Angles and Tees in Tension*

4.2.1.1 In the case of single angle connected through one leg the net effective sectional area shall be taken as:

$$A_1 + A_2 k$$

where

A_1 = effective cross-sectional area of the connected leg,

A_2 = the gross cross-sectional area of the unconnected leg, and

$$k = \frac{3A_1}{3A_1 + A_2}$$

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

4.2.1.2 In the case of a pair of angles back-to-back (or a single tee) connected by one leg of each angle (or by the flange of the tee) to the same side of a gusset, the net effective area shall be taken as

$$A_1 + A_2 k$$

where

A_1 and A_2 are as defined in **4.2.1.1**, and

$$k = \frac{5A_1}{5A_1 + A_2}$$

The angles shall be connected together along their length in accordance with the requirements under **8.10.3.3**.

4.2.1.3 For 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 sections the areas to be taken in computing the mean tensile stress shall be the effective area provided the members are connected together along their length as specified in **8.10.3.3**

4.2.1.4 Where the angles are back-to-back but are not tack riveted or welded according to **8.10.3.3** the provisions under **4.2.1.2** and **4.2.1.3** shall not apply and each angle shall be designed as a single angle connected through one leg only in accordance with **4.2.1.1**.

4.2.1.5 When two tees are placed back-to-back but are not tack riveted or welded as per **8.10.3.3** the provisions under **4.2.1.3** shall not apply and each tee shall be designed as a single tee connected to one side of a gusset only in accordance with **4.2.1.2**.

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

SECTION 5 DESIGN OF COMPRESSION MEMBERS

5.1 Axial Stresses in Uncased Struts

5.1.1 The direct stress in compression on the gross sectional area of axially loaded compression members shall not exceed $0{\cdot}6 f_y$ nor the permissible stress σ_{ac}, calculated using the following formula:

$$\sigma_{ac} = 0{\cdot}6 \frac{f_{cc} \cdot f_y}{[(f_{cc})^n + (f_y)^n]^{1/n}}$$

where

σ_{ac} = permissible stress in axial compression, in MPa;

f_y = yield stress of steel, in MPa;

f_{cc} = elastic critical stress in compression, $= \dfrac{\pi^2 . E}{\lambda^2}$;

E = modulus of elasticity of steel; 2×10^5 MPa;

$\lambda\ (= l/r)$ = slenderness ratio of the member, ratio of the effective length to appropriate radius of gyration; and

n = a factor assumed as 1·4.

Values of σ_{ac} for some of the Indian Standard structural steels are given in Table 5.1 for convenience.

5.2 Effective Length of Compression Members

5.2.1 *General* — The slenderness ratio of a strut shall be calculated as the ratio of the effective length, l, to the appropriate radius of gyration, r. The effective length, l shall be derived from the actual length, L. The actual strut length shall be taken as the length from the centre-to-centre of

TABLE 5.1 PERMISSIBLE STRESS σ_{ac} (MPa N AXIAL COMPRESSION FOR STEELS WITH VARIOUS YIELD STRESS

(*Clause* 5.1.1)

$f_y \rightarrow$ / $\lambda \downarrow$	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	132	138	144	150	156	168	180	192	204	215	227	239	251	269	287	305	323
20	131	137	142	148	154	166	177	189	201	212	224	235	246	263	280	297	314
30	128	134	140	145	151	162	172	183	194	204	215	225	236	251	266	280	295
40	124	129	134	139	145	154	164	174	183	192	201	210	218	231	243	255	267
50	118	123	127	132	136	145	153	161	168	176	183	190	197	207	216	225	233
60	111	115	118	122	126	133	139	146	152	158	163	168	173	180	187	193	199
70	102	106	109	112	115	120	125	130	135	139	142	147	150	155	160	164	168
80	93	96	98	101	103	107	111	115	118	121	124	127	129	133	136	139	141
90	85	87	88	90	92	95	98	101	103	105	108	109	111	114	116	118	119
100	76	78	79	80	82	84	86	88	90	92	93	94	96	97	99	100	101
110	68	69	71	72	73	74	76	77	79	80	81	82	83	84	85	86	87
120	61	62	63	64	64	66	67	67	69	70	71	71	72	73	73	74	75
130	55	55	56	57	57	58	59	60	61	61	62	62	63	63	64	64	65
140	49	50	50	51	51	52	53	53	54	54	54	55	55	56	56	56	57
150	44	45	45	45	46	46	47	47	48	48	48	49	49	49	49	50	50
160	40	40	41	41	41	42	42	42	43	43	43	43	43	44	44	44	44
170	36	36	37	37	36	37	38	38	38	38	39	39	39	39	39	39	39
180	33	33	33	33	33	34	34	34	34	35	35	35	35	35	35	35	35
190	30	30	30	30	30	30	31	31	31	31	31	31	32	32	32	32	32
200	27	27	28	28	28	28	28	28	28	28	28	28	28	28	28	28	28
210	25	25	25	25	25	25	26	26	26	26	26	26	26	26	26	26	26
220	23	23	23	23	23	23	23	24	24	24	24	24	24	24	24	24	24
230	21	21	21	21	21	21	22	21	22	22	22	22	22	22	22	22	22
240	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
250	18	18	18	18	18	18	18	18	18	19	19	19	19	19	19	19	19

inter-sections with supporting members, or the cantilevered length in the case of free-standing struts.

5.2.2 *Effective Length* — Where accurate frame analysis is not done, the effective length of a compression member in a given plane may be determined by the procedure given in Appendix C. However, in most cases the effective length in the given plane assessed on the basis of Table 5.2, would be adequate. Effective length as given in Table 5.2 may also be adopted where columns directly form part of framed structures.

5.2.3 *Eccentric Beam Connections* — In cases where the beam connections are eccentric with respect to the axes of the columns, the same conditions of restraint shall be deemed to apply, provided the connections are carried across the flange or web of the columns as the case may be, and the web of the beam lies within, or in direct contact with the column section. Where practical difficulties prevent this, the effective length shall be estimated to accord with the case appropriate to no restraint in that direction.

5.2.4 *Members of Trusses* — In the case of bolted, riveted or welded trusses and braced frames, the effective length 'l' of the compression members shall be taken as between 0·7 and 1·0 times the distance between centres of inter-sections, depending on the degree of end restraint provided. In the case of members of trusses buckling in the plane perpendicular to the plane of the truss the effective length shall be taken as 1·0 times the distance between points of restraints. The design of discontinuous angle struts shall be as specified in **5.5**.

5.2.5 *Stepped Columns* — A method of determining the effective length of stepped columns is given in Appendix D.

5.3 Design Details

5.3.1 *Thickness of Elements* — The thickness of an outstanding leg of any member in compression shall be in accordance with **3.5.2.1** and **3.5.2.2**.

5.3.2 *Effective Sectional Area* — Except as modified under **3.5.2** the gross sectional area shall be taken for all compression members connected by welds and turned and fitted bolts and pins except that holes, which are not fitted with rivets, weld or tight-fitting bolts and pins, shall be deducted.

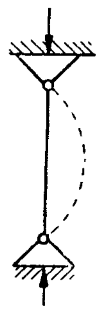
5.3.3 *Eccentricity for Stanchion and Solid Columns*

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

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

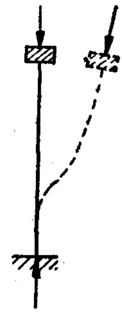
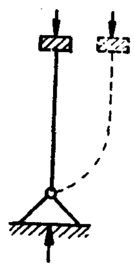
TABLE 5.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS OF CONSTANT DIMENSIONS

(*Clause* 5.2.2)

Degree of End Restraint of Compression Member	Recommended Value of Effective Length	Symbol
(1)	(2)	(3)
a) Effectively held in position and restrained against rotation at both ends	0·65 L	
b) Effectively held in position at both ends and restrained against rotation at one end	0·80 L	
c) Effectively held in position at both ends, but not restrained against rotation	1·00 L	

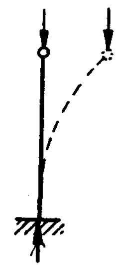
(*Continued*)

TABLE 5.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS OF CONSTANT DIMENSIONS — *Contd*

DEGREE OF END RESTRAINT OF COMPRESSION MEMBER	RECOMMENDED VALUE OF EFFECTIVE LENGTH	SYMBOL
(1)	(2)	(3)
d) Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position	1·20 L	
e) Effectively held in position and restrained against rotation at one end, and at the other end partially restrained against rotation but not held in position	1·50 L	
f) Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position	2·00 L	

(*Continued*)

TABLE 5.2 EFFECTIVE LENGTH OF COMPRESSION MEMBERS OF CONSTANT DIMENSIONS — *Contd*

DEGREE OF END RESTRAINT OF COMPRESSION MEMBER	RECOMMENDED VALUE OF EFFECTIVE LENGTH	SYMBOL
(1)	(2)	(3)
g) Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end	2·00 L	

NOTE 1 — L is the unsupported length of compression member.

NOTE 2 — For battened struts the effective length shall be increased by 10 percent.

b) 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.

5.3.3.2 In continuous columns, the bending moments due to eccentricities of loading on the columns at any floor may be taken as:

a) ineffective at the floor levels above and below that floor; and

b) divided equally between the column's lengths above and below that floor level, provided that the moment of inertia of either column section, divided by its effective length does not exceed 1·5 times the corresponding value of the other column. In case where this ratio is exceeded, the bending moment shall be divided in proportion to the moments of inertia of the column sections divided by their respective effective lengths.

5.3.4 *Splices*

5.3.4.1 Where the ends of compression members are faced for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist any tension when bending is present.

The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

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

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

5.4 Column Bases

5.4.1 *Gusseted Bases*—For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc, in combination with the bearing area of the shaft shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified stresses. All the bearing surfaces shall be machined to ensure perfect contact.

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

5.4.2 *Column and Base Plate Connections* — Where the end of the column is connected directly to the base plate by means of full penetration butt welds the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected.

5.4.3 *Slab Bases* — Columns with slab bases need not be provided with gussets, but fastenings shall be provided sufficient to retain the parts securely in plate and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. When the slab alone distributes the load uniformly, the minimum thickness of a rectangular slab shall be given by the following formula:

$$t = \sqrt{\frac{3\,w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

where

t = the slab thickness, in mm;

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

a = the greater projection of the plate beyond column, in mm;

b = the lesser projection of the plate beyond the column, in mm; and

σ_{bs} = the permissible bending stress in slab bases (for all steels, shall be assumed as 185 MPa).

5.4.3.1 When the slab does not distribute the loading uniformly or where the slab is not rectangular, special calculations shall be made to show that the stresses are within the specified limits.

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

$$t = 10 \sqrt{\frac{90\,W}{16\,\sigma_{bs}} \times \frac{B}{B - d_0}}$$

where

t = the thickness of the plate, in mm;

W = the total axial load, in kN;

B = the length of the side of cap or base, in mm;

σ_{bs} = the permissible bending stress in slab bases (for all steels, shall be assumed as 185 MPa); and

d_0 = the diameter of the reduced end, if any, of the column, in mm.

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

5.4.3.4 The cap or base plateshall not be less than 1·5(d_0 + 75) mm in length or diameter.

5.4.3.5 The area of the shoulder (the annular bearing area) shall be sufficient to limit the stress in bearing, for the whole of the load communicated to the slab, to the maximum values given in **6.3**, and resistance to any bending communicated to the shaft by the slab shall be taken as assisted by bearing pressures developed against the reduced end of the shaft in conjunction with the shoulder.

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

5.4.3.7 In cases where the cap or base is fillet welded direct to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required in **5.4.3** and its sub-clauses for fastening to slab bases. Where full strength T-butt welds are provided no machining of contact surfaces shall be required.

5.4.4 *Base Plates and Bearing Plates* — The base plates and grillages of stanchions and the bearing and spreaders of beams and girders shall be of adequate strength, stiffness and area, to spread the load upon the concrete, masonry, other foundation, or other supports without exceeding the permissible stress on such foundation under any combination of load and bending moments.

5.5 Angle Struts

5.5.1 *Single Angle Struts*

a) Single angle discontinuous struts connected by a single rivet or bolt may be designed for axial load only provided the compressive stress does not exceed 80 percent of the values given in Table 5.1 in which the effective length 'l' of the strut shall be taken as centre-to-centre of intersection at each end and 'r' is the minimum radius of gyration. In no case, however, shall the ratio of slenderness for such single angle struts exceed 180.

b) Single angle discontinuous struts connected by a weld or by two or more rivets or bolts in line along the angle at each end may be designed for axial load only provided the compression stress does not exceed the values given in Table 5.1, in which the effective length 'l' shall be taken as 0·85 time the length of the strut, centre-to-centre of intersection at each end and 'r' is the minimum radius of gyration.

5.5.2 *Double Angle Struts*

a) For double angle discontinuous struts, back to back connected to both sides of the gusset or section by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length 'l' in the plane of end gusset shall be taken as between 0·7 and 0·85 times the distance between intersections, depending on the degree of the restraint provided and in the plane perpendicular to that of the end gusset, the effective length 'l' shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values obtained from Table 5.1 for the ratio of slenderness based on the appropriate radius of gyration. The angles shall be connected tegether in their lengths so as to satisfy the requirements of **5.9** and **8.10.3**.

b) Double angle discontinuous struts back-to-back, connected to one side of a gusset or section by a one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed as for single angles in accordance with **5.5.1** (a) and the angles shall be connected together in their length so as to satisfy the requirements of **5.9** and **8.10.3**.

5.5.3 *Continuous Members* — Single or double angle continuous struts such as those forming the flanges, chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with **5.2.4**.

5.5.4 *Combined Stresses* — If the struts carry, in addition to axial loads, loads which cause transverse bending, the combined bending and axial stresses shall be checked in accordance with **7.1.1**. For determining the permissible axial and bending stresses, for use in applying **7.1.1**, the effective length shall be taken in accordance with **5.2** and **6.6.1**, respectively.

5.6 Steel Castings — The use of steel castings shall be limited to bearings, junctions and other similar parts and the working stresses shall not exceed the workings stresses given in this standard for steel of yield stress 250 MPa.

5.7 Lacing

5.7.1 *General*

5.7.1.1 Compression members comprising of two main components laced and tied should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis in the plane of lacing (*see* Fig. 5.1A).

5.7.1.2 As far as practicable the lacing system shall not be varied throughout the length of the strut.

5.7.1.3 Except for tie plates as specified in **5.8** double laced system (*see* Fig. 5.1B) and single laced systems on opposite sides of the main components shall not be combined with cross members perpendicular to the longitudinal axis of the strut unless all forces resulting from deformation of the strut members are calculated and provided for in the lacing and its fastenings (*see* Fig. 5.1C).

5.7.1.4 Single laced systems on opposite sides of the components shall preferably be in the same direction so that one be the shadow of the other, instead of being mutually opposed in direction (*see* Fig. 5.1D).

5.7.2 *Design of Lacing*

5.7.2.1 The lacing of compression members shall be proportioned to resist a total transverse shear 'V' at any point in the length of the member equal to at least 2·5 percent of the axial force in the member, which shear shall be considered as divided equally among all transverse lacing systems in parallel planes.

5.7.2.2 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the shear due to the bending in addition to that specified under **5.7.2.1**.

5.7.2.3 The slenderness ratio 'λ' of the lacing bars for compression members shall not exceed 145. In riveted construction, the effective length of lacing bars for the determination of the permissible stress shall be taken as the length between the inner end rivets of the bars for single lacing, and as 0·7 of this length for double lacing effectively riveted at intersection. In welded construction, the effective lengths shall be taken as 0·7 times the distance between the inner ends of welds connecting the lacing bars to the member.

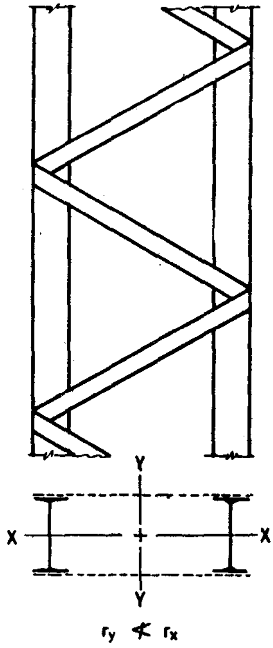


Fig. 5.1A Lacing Details

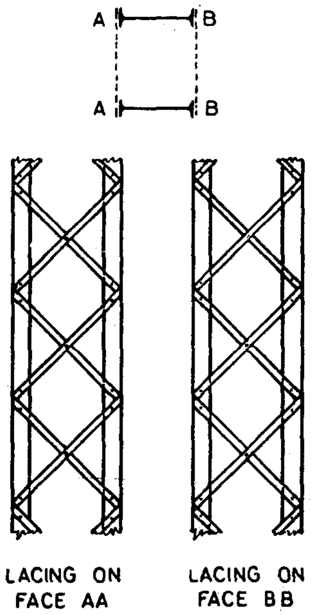


Fig. 5.1B Double Lacing System

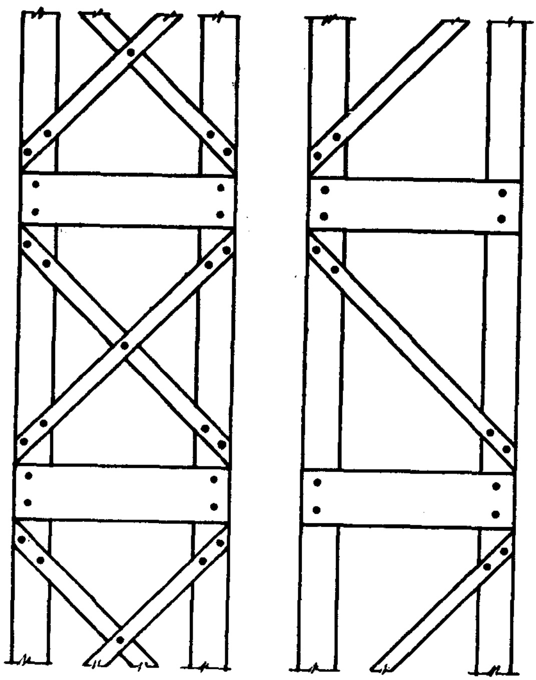


FIG. 5.1C DOUBLE LACED AND SINGLE LACED SYSTEMS COMBINED WITH CROSS MEMBERS

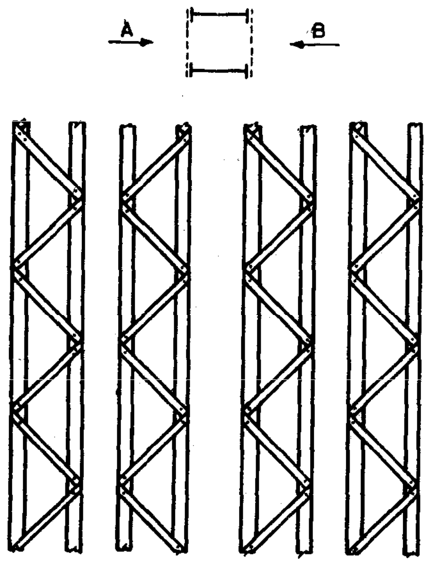


FIG. 5.1D SINGLE LACED SYSTEM ON OPPOSITE SIDES OF MAIN COMPONENTS

5.7.3 *Width of Lacing Bars.*— In riveted construction, the minimum width of lacing bars shall be as follows:

Nominal Rivet Dia	*Width of Lacing Bars*
mm	mm
22	65
20	60
18	55
16	50

5.7.4 *Thickness of Lacing Bars* — The thickness of flat lacing bars shall be not less than one-fortieth of the length between the inner end rivets or welds for single lacing, and one-sixtieth of this length for double lacing riveted or welded at intersections.

5.7.4.1 Rolled sections or tubes of equivalent strength may be used instead of flats.

5.7.5 *Angle of Inclination* — Lacing bars, whether in double or single systems, shall be inclined at an angle not less than 40 degree nor more than 70 degrees to the axis of the member.

NOTE — The required section for lacing bars for compression members or for tension members subject to bending shall be determined by using the appropriate permissible stresses subject to the requirements in **5.7.3** and **5.7.4**. For tension members under stress, only the lacing bars shall be subject to the requirements of **5.7.3**, **5.7.4** and **5.7.5**.

5.7.6 *Spacing*

5.7.6.1 The maximum spacing of lacing bars, whether connected by riveting or welding, shall also be such that the minimum slenderness ratio λ ($=l/r$) of the components of the member 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 less, where 'l' is the distance between the centres of connection of the lattice bars to each component.

5.7.6.2 Where lacing bars are not lapped to form the connection to the components of the members, they shall be so connected that there is no appreciable interruption in the triangulation of the system.

5.7.7 *Attachment to Main Members* — The riveting or welding of lacing bars to the main members shall be sufficient to transmit the load in the bars. Where welded lacing bars overlap the main members, the amount of lap measured along either edge of the lacing bar shall be not less than four times the thickness of the bar or the members, whichever is less. The welding should be sufficient to transmit the load in the bar and shall, in any case, be provided along each side of the bar for the full length of lap.

5.7.7.1 Where lacing bars are fitted between the main members, they shall be connected to each member by fillet welds on each side of the bar or by full penetration butt welds. The lacing bars shall be so placed as to be generally opposite the flange or stiffening elements of the main member.

5.7.8 *End Tie Plates* — Laced compression members shall be provided with tie plates at the ends of lacing systems and at points where the systems are interrupted (*see also* **5.8**).

5.8 Battening and Tie Plates

5.8.1 *General*

5.8.1.1 Compression members composed of two main components battened should preferably have their two main components of the same cross section and symmetrically disposed about their *x-x* axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis in the plane of batten.

5.8.1.2 Battened compression members not complying with the requirements specified in this clause or those subjected, in the plane of the battens, to eccentricity of loading, applied moments or lateral forces (*see* Fig. 5.2) shall be designed according to the exact theory of elastic stability or empirically from the verification of tests, so that they have a load factor of not less than 1·7 in the actual structure.

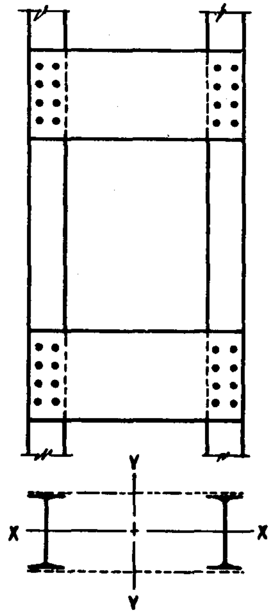


FIG. 5.2 BATTEN COLUMN SECTION

NOTE — If the column section is subjected to eccentricity or other moments about *y-y* axis the battens and the column section should be specially designed for such moments.

5.8.1.3 The battens shall be placed opposite each other at each end of the member and points where the member is stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length from centre to centre of connection.

5.8.2 *Design*

5.8.2.1 *Battens* — Battens shall be designed to carry the bending moments and shears arising from transverse shear force 'V' of 2·5 percent 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. The main members shall also be checked for the same shear force and bending moments as for the battens.

Battens shall be of plates, angles, channels, or I-sections and shall be riveted or welded to the main components so as to resist simultaneously a longitudinal shear $V_1 = \frac{VC}{N.S}$ and a moment $M = \frac{V.C}{2N}$

where

V = the transverse shear force as defined above;

C = the distance centre-to-centre of battens, longitudinally;

N = the number of parallel planes of battens; and

S = the minimum transverse distance between the centroids of the rivet group/welding.

5.8.2.2 *Tie plates* — Tie plates shall be designed by the same method as battens. In no case shall a tie plate and its fastenings be incapable of carrying the forces for which the lacing has been designed.

5.8.2.3 *Size* — When plates are used for battens, the end battens and those at points where the member is stayed in its length shall have an effective depth, longitudinally, of not less than the perpendicular distance between the centroids of the main members, and intermediate battens shall have an effective depth of not less than three quarters of this distance, but in no case shall the effective depth of any batten be less than twice the width of one member in the plane of the battens. The effective depth of a batten shall be taken as the longitudinal distance between end rivets or end welds.

The thickness of batten or the tie plates shall be not less than one-fiftieth of the distance between the innermost connecting lines of rivets or welds.

5.8.2.4 The requirement of size and thickness specified above does not apply when angles, channels or I-sections are used for battens with their legs or flanges perpendicular to the main member. However, it should be ensured that the ends of the compression members are tied to achieve adequate rigidity.

5.8.3 *Spacing of Battens*

5.8.3.1 In battened compression members not specifically checked for shear stress and bending moments as specified in **5.8.2.1**, the spacing of battens centre-to-centre of end fastenings shall be such that the slenderness ratio 'λ' of the lesser main component over that distance shall be not greater than 50 or greater than 0·7 time the slenderness ratio of the member as a whole, about its x-x (axis parallel to the battens).

NOTE — With regard to effective length of the battened compression member as a whole, reference may be made to Table 5.2.

5.8.3.2 The number of battens shall be such that the member is divided into not less than three parts longitudinally.

5.8.4 *Attachment to Main Members*

5.8.4.1 *Welded connections* — Where tie or batten plates overlap the main members, the amount of lap shall be not less than four times the thickness of the plate. The length of weld connecting each edge of the batten plate to the member shall, in aggregate, be not less than half the depth of the batten plate. At least one-third of the weld shall be placed at each end of this edge. The length of weld and depth of batten plate shall be measured along the longitudinal axis of the main member.

In addition, the welding shall be returned along the other two edges of the plates transversely to the axis of the main member for a length not less than the minimum lap specified above.

5.9 Compression Members Composed of Two Components Back-to-Back

5.9.1 Compression members composed of two angles, channels, or tees, back-to-back in contact or separated by a small distance shall be connected together by riveting, bolting or welding so that the ratio of slenderness 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 less (*see also* Section 8).

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

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

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

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

5.9.5 Where the components are in contact back-to-back, the spacing of the rivets, bolts or intermittent welds shall not exceed the maximum spacing for compression members as given in **6.1.4** and **6.2.6** of IS : 816-1969.

SECTION 6 DESIGN OF MEMBERS SUBJECTED TO BENDING

6.1 General — The calculated stress in a member subjected to bending shall not exceed any of the appropriate maximum permissible stresses given in **6.2** for bending, **6.3** for bearing, **6.4** for shear and in **7.1** for the combination of stresses.

6.2 Bending Stresses

6.2.1 *Maximum Bending Stresses* — The maximum bending stress in tension ($\sigma_{bt,\,cal}$) or in compression ($\sigma_{bc,\,cal}$) in extreme fibre calculated on the effective section of a beam shall not exceed the maximum permissible bending stress in tension (σ_{bt}) or in compression (σ_{bc}) obtained as follows nor the values specified in **6.2.2**, **6.2.3**, **6.2.5** and **6.2.6**, as appropriate:

$$\sigma_{bt} \text{ or } \sigma_{bc} = 0{\cdot}66 f_y .$$

6.2.2 *Maximum Permissible Bending Compressive Stress in Beams and Channels with Equal Flanges* — For an I-beam or channel with equal flanges bent about the axis of maximum strength (*x-x* axis), the maximum bending compressive stress on the extreme fibre calculated on the effective section shall not exceed the values of maximum permissible bending compressive stress, σ_{bc}, given directly in Table 6.1A or 6.1B, Table 6 1C or 6.1D and Table 6.1E or 6.1F, as appropriate, for steels with yield stress f_y of 250 MPa, 340 MPa and 400 MPa, respectively. For steels with yield stresses other than those covered in Tables 6.1A to 6.1F, maximum permissible bending compressive stress shall be obtained in accordance with **6.2.3** and **6.2.4.**

NOTE — Tables 6.1A to 6.1F have been derived in accordance with **6.2.3** and **6.2.4**.

6.2.2.1 In Tables 6.1A to 6.1F:

D = overall depth of beam;

d_1 = depth of web (*see* **1.3**);

l = effective length of compression flange (*see* **6.6**);

r_y = radius of gyration of the section about its axis of minimum strength (*y-y* axis);

T = mean thickness of the compression flange, is equal to the area of horizontal portion of flange divided by width; and

t = web thickness.

For rolled sections, the mean thickness is that given in appropriate Indian Standards.

In case of compound girders with curtailed flanges, D shall be taken as the overall depth of the girder at the point of maximum bending moment, and T shall be taken as the effective thickness of the compression flange and shall be calculated as:

$T = K_1 \times$ mean thickness of the horizontal portion of the compression flange at the point of maximum bending moment. Coefficient K_1 is defined in **6.2.4**.

6.2.3 *Maximum Permissible Bending Compressive Stress in Beams and Plate Girders* — For beams and plate girders, bent about the axis of maximum strength (*x-x* axis), the maximum bending compressive stress on the extreme fibre, calculated on the effective section shall not exceed the maximum permissible bending compressive stress σ_{bc} in MPa obtained by the following formula:

$$\sigma_{bc} = 0.66 \frac{f_{cb} \cdot f_y}{[(f_{cb})^n + (f_y)^n]^{1/n}}$$

where

f_{cb} = elastic critical stress in bending, calculated in accordance with **6.2.4** or by an elastic flexural-torsional buckling analysis, in MPa;

f_y = yield stress of the steel in MPa; and

n = a factor assumed as **1.4**.

Values of σ_{bc} as derived from the above formula for some of the Indian Standard structurel steels are given in Table 6.2.

TABLE 6.1A MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL-FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with $f_y = 250$ MPa, $\frac{T}{t} > 2{\cdot}0$ or $\frac{d_1}{t} > 85$

$D/T \rightarrow$ / $l/r_y \downarrow$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	160	160	159	159	158	158	158	158	158	157	157	157	157	157	157
45	159	158	157	157	156	156	156	155	155	155	155	155	154	154	154
50	158	157	156	155	154	154	153	153	152	152	152	151	151	151	151
55	157	155	154	153	152	151	150	149	149	148	148	148	148	147	147
60	156	153	152	150	149	148	148	146	145	145	144	144	144	143	143
65	154	152	150	148	147	145	144	143	142	141	140	140	139	139	139
70	153	150	148	146	144	142	141	139	138	137	136	135	135	135	134
75	152	148	145	143	141	139	138	136	134	133	132	131	130	130	129
80	150	147	143	141	138	136	135	132	130	128	128	126	126	125	125
85	149	145	141	138	136	133	132	128	126	124	123	122	121	120	120
90	147	143	139	136	133	130	128	124	122	120	119	117	116	115	115
95	146	141	137	133	130	128	125	121	118	116	114	112	111	110	110
100	145	140	135	131	128	125	122	117	114	112	110	108	107	105	105
110	142	136	131	127	123	119	116	111	107	104	102	99	98	96	95
120	139	133	127	122	118	114	111	104	100	97	94	91	90	88	87
130	137	130	124	118	113	109	106	99	94	90	88	84	82	80	79
140	134	127	120	114	109	105	101	93	88	84	81	78	75	73	72
150	132	124	117	110	105	100	96	88	83	79	76	72	69	67	65
160	129	121	113	107	101	96	92	84	78	74	71	66	64	61	60
170	127	118	110	104	98	93	88	80	74	69	66	62	59	56	55
180	124	115	107	100	94	89	85	76	70	65	62	58	55	52	50
190	122	113	104	97	91	86	82	73	66	62	58	54	51	48	46
200	120	110	102	94	88	83	78	70	63	59	55	50	48	44	43
210	118	108	99	92	86	80	76	67	60	56	52	47	44	41	40
220	116	105	97	89	83	78	73	64	58	53	49	45	42	38	37
230	113	103	94	87	80	75	70	62	55	51	47	42	39	36	34
240	111	101	92	84	78	73	68	59	53	48	45	40	37	34	32
250	109	99	90	82	76	70	66	57	51	46	43	38	35	32	30
260	107	97	88	80	74	68	64	55	49	44	41	36	33	30	28
270	106	95	86	78	72	66	62	53	47	43	39	35	32	28	26
280	104	93	84	76	70	65	60	51	45	41	38	33	30	27	25
290	102	91	82	74	68	63	58	50	44	39	36	32	29	25	24
300	100	89	80	72	66	61	57	48	42	38	35	30	27	24	22

TABLE 6.1B MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with f_y = 250 MPa, $\frac{T}{t} \leqslant 2{\cdot}0$ and $\frac{d_1}{t} \leqslant 85$

D/T → l/r_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	161	161	160	160	160	160	160	159	159	159	159	159	159	159	159
45	161	160	159	159	158	158	158	157	157	157	157	157	157	157	157
50	160	158	158	157	156	156	156	155	155	155	154	154	154	154	154
55	159	157	156	155	154	154	153	153	152	152	152	151	151	151	151
60	158	156	154	153	152	152	151	150	149	149	149	148	148	148	148
65	156	154	153	151	150	149	148	147	146	146	145	145	144	144	144
70	155	153	151	149	149	147	146	144	143	142	142	141	141	140	140
75	154	152	149	147	146	144	143	141	140	139	138	137	137	136	136
80	153	150	148	145	143	142	140	138	136	135	134	133	132	132	132
85	152	149	146	143	141	139	138	135	133	131	130	129	128	127	127
90	151	147	144	141	139	137	135	131	129	127	126	125	124	123	123
95	150	146	142	139	137	134	132	128	126	124	122	121	120	119	118
100	149	145	141	137	134	132	129	125	122	120	118	116	115	114	113
110	147	142	137	133	130	127	124	119	115	113	111	108	107	105	105
120	144	139	134	129	126	122	119	113	109	106	104	101	99	97	96
130	142	136	131	126	121	118	114	108	103	99	97	94	91	89	88
140	140	133	128	122	118	113	110	103	97	94	91	87	85	82	81
150	138	131	124	119	114	109	105	98	92	88	85	81	78	76	74
160	136	128	121	115	110	106	101	93	87	83	80	75	73	70	68
170	134	126	119	112	107	102	98	89	83	79	75	70	68	64	63
180	131	123	116	109	104	99	94	85	79	74	71	66	63	60	58
190	129	121	113	106	101	95	91	82	75	71	67	62	59	55	54
200	127	118	111	104	98	92	88	79	72	67	63	58	55	51	50
210	125	116	108	101	95	90	85	76	69	64	60	55	52	48	46
220	123	114	106	99	92	87	82	73	66	61	57	52	49	45	43
230	122	112	103	96	90	84	80	70	63	58	55	49	46	42	40
240	120	110	101	94	87	82	77	68	61	56	52	47	43	40	38
250	118	108	99	92	85	80	75	65	59	54	50	44	41	37	35
260	116	106	97	89	83	77	73	63	57	52	48	42	39	35	33
270	114	104	95	87	81	75	71	61	55	50	46	41	37	33	31
280	113	102	93	85	79	73	69	59	53	48	44	39	35	32	30
290	111	100	91	84	77	72	67	58	51	46	42	37	34	30	28
300	109	98	89	82	75	70	65	56	49	45	41	36	32	29	27

TABLE 6.1 C MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with f_y = 340 MPa, $\frac{T}{t} > 2{\cdot}0$ or $\frac{d_1}{t} > 75$

D/T → l/r_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	215	214	212	212	211	211	210	210	209	209	209	209	209	209	209
45	213	211	209	208	207	206	206	205	204	204	204	203	203	203	203
50	210	208	205	204	203	202	201	199	199	198	198	197	197	197	197
55	208	204	202	200	198	197	196	194	193	192	191	191	190	190	190
60	205	201	198	195	193	191	190	188	186	185	185	184	183	183	183
65	203	198	194	191	188	186	185	181	180	178	177	176	176	175	175
70	200	195	190	186	183	181	179	175	173	171	170	169	168	167	167
75	198	192	186	182	179	176	173	169	166	164	163	161	160	159	159
80	195	188	183	178	174	170	168	163	159	157	156	154	153	151	151
85	193	185	179	174	169	165	162	157	153	150	149	146	145	144	143
90	190	182	175	169	165	161	157	151	147	144	142	139	138	136	136
95	188	179	172	165	160	156	152	145	141	137	135	132	131	129	128
100	185	176	168	162	156	151	147	140	135	131	129	126	124	122	121
110	180	170	162	154	148	143	138	130	124	120	117	114	112	109	108
120	176	165	155	147	141	135	130	121	115	110	107	103	101	98	97
130	171	159	149	141	134	128	122	113	106	101	98	93	91	88	87
140	167	154	144	135	127	121	116	105	98	93	90	85	82	79	78
150	163	150	139	129	122	115	110	99	92	87	83	78	75	72	70
160	158	145	134	124	116	110	104	93	86	80	77	72	68	65	63
170	155	141	129	120	111	105	99	88	80	75	71	66	63	59	58
180	151	137	125	115	107	100	94	83	76	70	66	61	58	54	53
190	147	133	121	111	103	96	90	79	72	66	62	57	54	50	48
200	144	129	117	107	99	92	86	75	68	62	58	53	50	46	44
210	140	125	113	103	95	88	83	72	64	59	55	50	46	43	41
220	137	122	110	100	92	85	79	69	61	56	52	47	43	40	38
230	134	119	107	97	89	82	76	66	58	53	49	44	41	37	35
240	131	116	104	94	86	79	74	63	56	51	47	42	38	35	33
250	128	113	101	91	83	76	71	61	53	48	44	39	36	32	31
260	125	110	98	88	80	74	68	58	51	46	42	37	34	31	29
270	122	107	95	86	78	72	66	56	49	44	41	36	32	29	27
280	120	105	93	83	76	69	64	54	47	42	39	34	31	27	25
290	117	102	90	81	73	67	62	52	46	41	37	32	29	26	24
300	115	100	88	79	71	65	60	51	44	39	36	31	28	25	23

TABLE 6.1 D MAXIMUM PERMISSIBLE BENDING STRESSES, (σ_{bc} MPa), IN EQUAL FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with f_y = 340 MPa, $\frac{T}{t} \leqslant 2.0$ and $\frac{d_1}{t} \leqslant 75$

$D/T \rightarrow$ / $l/r_y \downarrow$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	217	216	215	214	214	213	213	213	212	212	212	212	212	212	212
45	215	214	212	211	211	210	210	209	208	208	208	208	208	207	207
50	213	211	209	208	207	206	206	205	204	203	203	203	203	202	202
55	211	209	206	205	203	202	201	200	199	198	198	197	197	197	197
60	209	206	203	201	199	198	197	195	193	193	192	191	191	191	190
65	207	203	200	197	195	193	192	189	188	187	186	185	184	184	184
70	205	201	197	194	191	189	187	184	182	181	180	178	178	177	177
75	203	198	194	190	187	184	182	178	176	174	173	172	171	170	169
80	201	195	190	186	183	180	177	173	170	168	167	165	164	163	162
85	199	193	187	183	179	175	173	168	164	162	160	158	157	156	155
90	197	190	184	179	175	171	167	162	158	156	154	151	150	148	148
95	195	187	181	175	171	167	163	157	153	150	148	145	143	142	141
100	193	185	178	172	167	163	159	152	147	144	142	138	137	135	134
110	188	180	172	165	159	155	150	142	137	133	130	126	124	122	121
120	184	175	166	159	152	147	142	133	127	123	120	116	113	110	109
130	180	170	161	153	146	140	135	125	119	114	110	106	103	100	99
140	177	165	156	147	140	134	128	118	111	106	102	97	94	91	89
150	173	161	151	142	134	128	122	112	104	99	95	89	86	83	81
160	169	157	146	137	129	122	117	106	98	92	88	82	79	75	74
170	166	153	142	132	124	117	111	100	92	86	82	76	73	69	67
180	162	149	137	128	120	113	107	95	87	81	77	71	67	63	61
190	159	145	133	124	115	108	102	91	82	76	72	66	63	59	56
200	155	141	130	120	111	104	98	86	78	72	68	62	58	54	52
210	152	138	126	116	108	100	94	83	74	69	64	58	54	50	48
220	149	135	123	113	104	97	91	79	71	65	61	55	51	47	45
230	146	132	119	109	101	94	88	76	68	62	58	52	48	44	42
240	143	128	116	106	98	91	85	73	65	59	55	49	45	41	39
250	141	126	113	103	95	88	82	70	62	57	52	46	43	38	36
260	138	123	110	100	92	85	79	68	60	54	50	44	40	36	34
270	135	120	108	98	89	82	77	65	58	52	48	42	38	35	32
280	133	117	105	95	87	80	74	63	56	50	46	40	36	32	30
290	130	115	103	93	84	78	72	61	54	48	44	38	35	31	29
300	128	112	100	90	82	76	70	59	52	46	42	37	33	29	27

TABLE 6.1 E MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with $f_y = 400$ MPa, $\frac{T}{t} > 2.0$ or $\frac{d_1}{t} > 67$

$D/T \rightarrow$ / $l/r_y \downarrow$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	250	248	247	245	245	244	243	243	242	242	242	241	241	241	241
45	247	244	242	240	239	238	237	236	235	235	234	234	234	233	233
50	244	240	237	234	233	231	230	228	227	227	226	226	225	225	225
55	240	235	232	229	226	224	223	221	219	218	217	216	216	216	215
60	236	231	226	223	220	217	216	212	210	209	208	207	206	206	205
65	233	226	221	217	213	210	208	204	202	200	199	197	197	196	195
70	229	222	216	211	207	203	201	196	193	191	189	188	187	186	185
75	226	217	211	205	200	196	193	188	184	182	180	178	177	175	175
80	222	213	206	199	194	190	186	180	176	173	171	168	167	166	165
85	219	209	201	194	188	183	179	172	167	164	162	159	158	156	155
90	216	205	196	188	182	177	173	165	160	156	154	151	149	147	146
95	212	201	191	183	177	171	166	158	152	149	146	142	140	138	137
100	209	197	187	178	171	165	160	151	145	141	138	135	133	130	129
110	203	189	178	169	161	155	149	139	133	128	125	121	118	115	114
120	196	182	170	160	152	145	140	129	121	116	113	108	106	103	101
130	191	176	163	153	144	137	131	119	112	106	103	98	95	92	90
140	185	169	156	146	137	129	123	111	103	98	94	88	85	82	80
150	179	163	150	139	130	122	116	104	96	90	86	81	77	74	72
160	174	158	144	133	124	116	109	97	89	83	79	74	70	67	65
170	169	152	139	127	118	110	104	92	83	78	73	68	64	61	59
180	165	147	134	122	113	105	97	86	78	72	68	63	59	55	54
190	160	143	129	117	108	100	94	82	74	68	64	58	55	51	49
200	156	138	124	113	104	96	90	78	70	64	60	54	51	47	45
210	152	134	120	109	100	92	86	74	66	60	56	51	47	43	41
220	148	130	116	105	96	88	82	71	63	57	53	47	44	40	38
230	144	126	112	101	92	85	79	67	60	54	50	45	41	37	36
240	141	123	109	98	89	82	76	65	57	52	47	42	39	35	33
250	137	119	106	95	86	79	73	62	54	49	45	40	37	33	31
260	134	116	103	92	83	76	70	60	52	47	43	38	35	31	29
270	131	113	100	89	81	74	68	57	50	45	41	36	33	29	27
280	128	110	97	86	78	71	66	55	48	43	39	34	31	27	26
290	125	107	94	84	76	69	64	53	46	41	38	33	30	26	24
300	122	105	92	82	74	67	62	52	45	40	36	31	28	25	23

TABLE 6.1 F MAXIMUM PERMISSIBLE BENDING STRESSES, σ_{bc} (MPa), IN EQUAL FLANGE I-BEAMS OR CHANNELS

(*Clause* 6.2.2)

with f_y = 400 MPa, $\frac{T}{t} \leqslant 2.0$ and $\frac{d_1}{t} \leqslant 67$

$D/T \rightarrow$ $l/r_y \downarrow$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
40	253	252	250	249	249	248	248	247	247	246	246	246	246	246	246
45	251	248	246	245	244	243	243	242	241	241	240	240	240	240	239
50	248	245	242	240	239	238	237	235	234	234	233	233	233	232	232
55	245	241	238	236	234	232	231	229	227	227	226	225	225	225	224
60	242	237	234	231	228	226	225	222	220	219	218	217	217	216	216
65	239	234	229	225	222	220	218	215	212	211	210	209	208	207	207
70	236	230	225	220	217	214	212	207	205	203	202	200	199	198	198
75	233	226	220	215	211	208	205	200	197	195	193	191	190	189	188
80	230	223	216	210	206	202	199	193	189	186	185	182	181	180	179
85	227	219	212	205	200	196	192	186	181	178	176	174	172	171	170
90	225	215	207	201	195	190	186	179	174	171	168	165	164	162	161
95	222	212	203	196	190	185	180	172	167	163	161	157	155	153	152
100	219	208	199	191	185	179	175	166	160	156	153	150	148	145	144
110	213	202	191	183	176	169	164	154	148	143	140	135	133	130	129
120	208	195	184	175	167	160	154	144	136	131	127	123	120	117	115
130	203	189	177	167	159	152	146	134	126	121	117	111	108	105	103
140	198	183	171	160	152	144	138	126	117	111	107	102	98	95	93
150	193	178	165	154	145	137	131	118	109	103	99	93	89	86	84
160	188	172	159	148	139	131	124	111	102	96	92	85	82	78	76
170	183	167	154	142	133	125	118	105	96	90	85	79	75	71	69
180	179	162	149	137	127	119	112	99	90	84	79	73	69	65	63
190	175	158	144	132	122	114	108	94	85	79	74	68	64	60	58
200	171	153	139	128	118	110	103	90	81	75	70	63	60	55	53
210	167	149	135	123	114	105	99	86	77	70	66	59	55	51	49
220	163	145	131	119	110	102	95	82	73	67	62	56	52	48	45
230	159	141	127	115	106	98	91	79	70	64	59	53	49	44	42
240	156	138	123	112	102	94	88	75	67	61	56	50	46	42	39
250	152	134	120	108	99	91	85	72	64	58	53	47	43	39	37
260	149	131	117	105	96	88	82	70	61	55	51	45	41	37	34
270	146	128	114	102	93	85	79	67	59	53	49	43	39	35	32
280	143	125	111	99	90	83	77	65	57	51	47	41	37	33	30
290	140	122	108	97	88	80	74	63	55	49	45	39	35	31	29
300	137	119	105	94	85	78	72	61	53	47	43	37	33	29	27

6.2.4 *Elastic Critical Stress* — If an elastic flexural analysis is not carried out, the elastic critical stress f_{cb} for beams and plate girders with I_y smaller than I_x shall be calculated using the following formula:

$$f_{cb} = k_1 \left(X + k_2 Y \right) \frac{c_2}{c_1}$$

where

$$X = Y \sqrt{\left[1 + \frac{1}{20} \left(\frac{lT}{r_y D} \right)^2 \right]} \text{ MPa}$$

$$Y = \frac{26{\cdot}5 \times 10^5}{(l/r_y)^2} \text{ MPa}$$

k_1 = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraint and depends on ψ, 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 such points of restraint. Values of k_1 for different values of ψ are given in Table 6.3.

$\left.\begin{matrix} l, r_y \\ D, T \end{matrix}\right\}$ = as defined in **6.2.2.1**.

k_2 = a coefficient to allow for the inequality of flanges, and depends on ω, the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the *y-y* axis of the girder, at the point of maximum bending moment. Values of k_2 for different values of ω are given in Table 6.4.

c_1, c_2 = respectively the lesser and greater distances from the section neutral axis to the extreme fibres.

I_y = moment of inertia of the whole section about the axis lying in the plane of bending (axis *y-y*), and

I_x = moment of inertia of the whole section about the axis normal to the plane of bending (*x-x* axis).

Values of X and Y are given in Table 6.5 for appropriate values of D/T and l/r_y.

TABLE 6.2 VALUES OF σ_{bc} CALCULATED FROM f_{cb} FOR DIFFERENT VALUES OF f_y

(*Clause* 6.2.3)

All units in MPa.

f_y → / f_{cb} ↓	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
20	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13
30	19	19	19	19	19	19	19	19	19	19	19	19	19	19	20	20	20
40	25	25	25	25	25	25	25	25	26	26	26	26	26	26	26	26	26
50	30	30	31	31	31	31	31	31	31	32	32	32	32	32	32	32	32
60	36	36	36	36	36	37	37	37	37	37	38	38	38	38	38	38	38
70	41	41	41	41	42	42	42	43	43	43	43	44	44	44	44	44	44
80	45	46	46	46	47	47	48	48	48	49	49	49	49	50	50	50	50
90	50	50	51	51	51	52	53	53	54	54	54	55	55	55	56	56	56
100	54	54	55	55	56	57	57	58	59	59	60	60	60	61	61	62	62
110	58	58	59	60	60	61	62	63	64	64	65	65	66	66	67	67	67
120	61	62	63	64	64	65	67	67	68	69	70	70	71	71	72	72	73
130	65	66	67	67	68	70	71	72	73	74	74	75	76	76	77	78	78
140	68	69	70	71	72	73	75	76	77	78	79	80	80	81	82	83	84
150	71	72	73	74	75	77	79	80	81	82	83	84	85	86	87	88	89
160	74	75	77	78	79	81	82	84	85	87	88	89	90	91	92	93	94
170	77	78	80	81	82	84	86	88	89	91	92	93	94	95	97	98	99
180	79	81	82	84	85	87	89	91	93	94	96	97	98	100	101	102	103
190	82	84	85	87	88	90	93	95	97	98	100	102	102	104	106	107	108

200	84	86	88	89	91	93	96	98	100	102	103	105	106	108	110	111	113
210	86	88	90	92	93	96	99	101	103	105	107	109	110	112	114	116	117
220	89	90	92	94	96	99	102	104	106	109	111	112	114	116	118	120	121
230	90	93	94	96	98	101	104	107	110	112	114	116	118	120	122	124	126
240	92	94	97	99	100	104	107	110	113	115	117	119	121	124	126	128	130
250	94	96	99	101	103	106	110	113	115	118	120	122	124	127	130	132	134
260	96	98	100	103	105	108	112	115	110	121	123	126	128	131	133	136	138
270	97	100	102	104	107	111	114	118	121	124	126	129	131	134	137	139	142
280	99	101	104	106	108	113	116	120	123	126	129	132	134	137	140	143	145
290	100	103	105	108	110	115	119	122	126	129	132	135	137	141	144	147	149
300	102	104	107	110	112	116	121	125	128	131	135	137	140	144	147	150	153
310	103	106	108	111	114	118	123	127	130	134	137	140	143	147	150	153	156
320	104	107	110	113	115	120	125	129	133	136	140	143	146	150	153	157	160
330	105	108	111	114	117	122	126	131	135	138	142	145	148	152	156	160	163
340	106	110	113	115	118	123	128	133	137	141	144	148	151	155	159	163	166
350	108	111	114	117	120	125	130	134	139	143	147	150	153	158	162	166	169
360	109	112	115	118	121	126	131	136	141	145	149	152	156	161	166	169	172
370	110	113	116	119	122	128	133	138	143	147	151	155	158	163	168	172	175
380	111	114	117	120	123	129	135	140	144	149	153	157	160	166	170	174	178
390	111	115	118	121	125	130	136	141	146	151	155	159	163	168	173	177	181
400	112	116	119	122	126	132	137	143	148	152	157	161	165	170	175	180	184
420	114	118	121	124	128	134	140	146	151	156	160	165	169	175	180	185	189
440	115	119	123	126	130	136	142	148	154	159	164	169	173	179	185	190	195

(*Continued*)

TABLE 6.2 VALUES OF σ_{bc} CALCULATED FROM f_{cb} FOR DIFFERENT VALUES OF f_y — *Contd*

All units in MPa.

f_y → / f_{cb} ↓	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
460	117	121	124	128	132	138	145	151	157	162	167	172	177	183	189	194	200
480	118	122	126	130	133	140	147	153	159	165	170	175	180	187	193	199	204
500	119	123	127	131	135	142	149	155	162	168	173	178	183	190	197	203	209
520	120	125	129	133	136	144	151	158	164	170	176	181	187	194	201	207	213
540	121	126	130	134	138	145	153	160	166	172	178	184	189	197	204	211	217
560	122	127	131	135	139	147	154	161	168	175	181	187	192	200	208	215	221
580	123	128	132	136	140	148	156	163	170	177	183	189	195	203	211	218	225
600	124	129	133	137	141	150	157	165	172	179	185	192	198	206	214	222	229
620	125	129	134	138	143	151	159	166	174	181	187	194	200	209	217	225	232
640	126	130	135	139	144	152	160	168	175	183	189	196	202	211	220	228	235
660	126	131	136	140	145	153	161	169	177	184	191	198	204	214	222	231	238
680	127	132	136	141	145	154	163	171	178	186	193	200	207	216	225	234	242
700	128	132	137	142	146	155	164	172	180	187	195	202	209	218	228	236	244
720	128	133	138	143	147	156	165	173	181	189	196	204	210	220	230	239	247
740	129	134	139	143	148	157	166	174	182	190	198	205	212	222	232	241	250
760	129	134	139	144	149	158	167	175	184	192	199	207	214	224	234	244	253
780	130	135	140	145	149	159	168	176	185	193	201	208	216	226	236	246	255
800	130	135	140	145	150	159	169	177	186	194	202	210	217	228	238	248	257
850	131	137	142	147	152	161	171	180	188	197	205	213	221	232	243	253	263
900	132	138	143	148	153	163	172	182	191	200	208	216	224	236	247	258	268
950	133	138	144	149	154	164	174	183	193	202	211	219	227	240	251	262	273
1 000	134	139	145	150	155	165	175	185	195	204	213	222	230	243	255	266	277

1 050	135	140	145	151	156	167	177	187	196	206	215	224	233	246	258	270	281
1 100	135	141	146	152	157	168	178	188	198	207	217	226	235	248	261	273	285
1 150	136	141	147	152	158	168	179	189	199	209	219	228	237	251	263	276	288
1 200	136	142	147	153	159	169	180	190	200	210	220	230	239	253	266	279	291
1 300	137	143	149	154	160	171	182	192	203	213	223	233	243	257	270	284	297
1 400	138	144	149	155	161	172	183	194	205	215	225	236	246	260	274	288	302
1 500	139	144	150	156	162	173	184	195	206	217	228	238	248	263	278	292	306
1 600	139	145	151	157	163	174	185	197	208	219	229	240	250	266	281	295	309
1 700	140	146	151	157	163	175	186	198	209	220	231	242	252	268	283	298	313
1 800	140	146	152	158	164	176	187	199	210	221	232	243	254	270	285	301	316
1 900	140	146	152	158	164	176	188	200	211	222	234	245	256	272	287	303	318
2 000	141	147	153	159	165	177	189	200	212	223	235	246	257	273	289	305	321
2 200	141	147	154	160	166	178	190	202	213	225	237	248	259	276	292	309	325
2 400	142	148	154	160	166	179	191	203	215	226	238	250	261	278	295	312	328
2 600	142	148	154	161	167	179	191	204	216	227	239	251	263	280	297	314	331
2 800	142	149	155	161	167	180	192	204	216	228	240	252	264	282	299	316	333
3 000	143	149	155	161	168	180	193	205	217	229	241	253	265	283	300	318	335
3 500	143	149	156	162	168	181	194	206	218	231	243	255	267	286	303	321	339
4 000	143	150	156	163	169	182	194	207	219	232	244	257	269	287	306	324	342
4 500	144	150	157	163	169	182	195	208	220	233	245	258	270	289	307	326	344
5 000	144	150	157	163	170	183	195	208	221	233	246	259	271	290	309	327	346
5 500	144	151	157	163	170	183	196	208	221	234	247	259	272	291	310	328	347
6 000	144	151	157	164	170	183	196	209	222	234	247	260	273	291	310	329	348

TABLE 6.3 VALUES OF k_1 FOR BEAMS WITH CURTAILED FLANGES

(*Clause* 6.2.4)

ψ	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2	0·1	0·0
k_1	1·0	1·0	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2

NOTE — Flanges should not be reduced in breadth to give a value of ψ lower than 0·25.

TABLE 6.4 VALUES OF k_2 FOR BEAMS WITH UNEQUAL FLANGES

(*Clause* 6.2.4)

ω	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2	0·1	0·0
k_2	0·5	0·4	0·3	0·2	0·1	0	−0·2	−0·4	−0·6	−0·8	−1·0

6.2.4.1 Values of f_{cb} shall be increased by 20 percent when T/t is not greater than 2·0 and d_1/t is not greater than 1 344/$\sqrt{f_y}$ where d_1 is as defined in **6.2.2.1** and **1.3** and t the thickness of web.

NOTE — Guidance for calculating elastic buckling forces may be found in the references listed in Appendix E.

6.2.5 *Beams Bent About the Axis of Minimum Strength (y-y axis)* — The maximum permissible bending stress in tension σ_{bt} or in compression σ_{bc} in beams bent about the axis of minimum strength shall not exceed 0·66 f_y, where f_y is the yield stress of steel.

6.2.6 *Angles and Tees* — The bending stress in the leg when loaded with the flange or table in compression shall not exceed 0·66 f_y. When loaded with the leg in compression, the permissible bending stress shall be calculated from **6.2.3** and **6.2.4** with $k_2 = -1·0$ and T = thickness of leg.

6.3 Bearing Stress — The bearing stress in any part of a beam when calculated on the net area of contact shall not exceed the value of σ_p determined by the following formula:

$$\sigma_p = 0·75 f_y$$

where

σ_p = maximum permissible bearing stress, and
f_y = yield stress of steel.

6.4 Shear Stresses

6.4.1 *Maximum Shear Stress* — The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, shall not exceed the value τ_{vm} given below:

$$\tau_{vm} = 0·45 f_y$$

where

τ_{vm} = maximum permissible shear stress, and

f_y = yield stress of steel.

6.4.2 *Average Shear Stress* — The average shear stress in a member calculated on the cross section of the web (*see* **6.4.2.1**) shall not exceed:

a) *For unstiffened webs* — the value τ_{va} obtained by the formula $\tau_{va} = 0{\cdot}4 f_y$, and

b) *For stiffened webs* — the values given in Tables 6.6A, 6.6B and 6.6C as appropriate for yield stress values 250, 340 and 400 MPa, respectively.

The values τ_{va} for stiffened webs for a steel whose yield stress is not given in Tables 6.6A, 6.6B and 6.6C shall be determined by using the following formulae, provided that the average stress τ_{va}, shall not exceed $0{\cdot}4 f_y$.

i) For webs where the distance between the vertical stiffeners is less than '*d*'

$$\tau_{va} = 0{\cdot}4 f_y \left[1{\cdot}3 - \frac{\sqrt{f_y} \cdot \frac{c}{t}}{4\,000 \left\{ 1 + \frac{1}{2} \left(\frac{c}{d} \right)^2 \right\}} \right]$$

ii) For webs where the distance between the vertical stiffeners is more than '*d*'

$$\tau_{va} = 0{\cdot}4 f_y \left[1{\cdot}3 - \frac{\sqrt{f_y} \cdot \frac{d}{t}}{4\,000 \left\{ 1 + \frac{1}{2} \left(\frac{d}{c} \right)^2 \right\}} \right]$$

where

τ_{va} = maximum permissible average shear stress.

c = distance between vertical stiffeners.

d =

1) *For vertically stiffened webs without horizontal stiffeners* — the clear distance between flange angles or, where there are no flange angles, the clear distance between flanges, ignoring fillets. Where tongue plates (*see* Fig. 6.1) having a thickness of not less than twice the thickness of the web plate are used, the depth *d* shall be taken as the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thickness of the tongue plates, whichever is less.

TABLE 6.5 VALUES OF X AND Y FOR CALCULATING f_{cb}

(*Clause* 6.2.4)

D/T → / l/r_y ↓	X															Y
	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40	2 484	2 222	2 066	1 965	1 897	1 849	1 814	1 759	1 728	1 709	1 697	1 683	1 675	1 667	1 663	1 656
45	2 103	1 856	1 708	1 612	1 546	1 499	1 465	1 411	1 380	1 362	1 349	1 335	1 327	1 319	1 315	1 309
50	1 822	1 590	1 449	1 357	1 293	1 248	1 214	1 161	1 131	1 113	1 101	1 086	1 078	1 070	1 067	1 060
55	1 607	1 389	1 254	1 166	1 105	1 061	1 028	976	947	929	917	902	894	886	883	876
60	1 437	1 232	1 104	1 020	961	918	886	835	806	788	776	762	754	746	743	736
65	1 301	1 107	985	904	847	806	775	726	697	679	667	653	645	637	634	627
70	1 188	1 005	889	811	757	717	687	638	610	592	581	567	559	551	547	541
75	1 094	920	810	735	682	644	615	567	540	522	511	497	489	481	478	471
80	1 014	849	743	672	621	584	556	509	482	465	454	440	432	424	421	414
85	945	788	687	618	570	533	506	461	434	417	406	392	385	377	373	367
90	886	735	639	573	526	491	464	420	394	377	366	353	345	337	334	327
95	833	689	597	534	488	454	428	385	360	343	332	319	311	304	300	294
100	787	649	560	499	455	423	398	356	331	314	304	290	283	275	272	265
110	708	582	499	443	402	371	347	307	283	268	257	244	237	229	226	219
120	644	527	451	398	359	330	308	270	247	232	222	209	202	194	191	184
130	591	482	411	361	325	298	277	240	218	204	194	181	174	167	163	157
140	546	444	378	331	297	271	251	217	195	181	172	160	153	145	142	135
150	508	412	350	306	274	249	230	197	177	163	154	142	135	145	124	118
160	474	385	326	284	254	230	212	181	161	148	139	127	121	113	110	104
170	445	360	305	265	236	214	197	167	148	135	126	115	109	102	98	92
180	420	339	286	249	221	200	184	155	137	125	116	105	98	92	88	82

190	397	320	270	235	208	188	172	145	127	115	107	96	90	83	80	73
200	376	304	256	222	197	177	162	136	119	107	99	89	83	76	73	66
210	358	288	243	210	186	168	153	128	112	101	93	82	76	70	66	60
220	341	275	231	200	177	159	145	121	105	94	87	77	71	64	61	55
230	326	262	220	191	169	152	138	115	99	89	82	72	66	60	56	50
240	312	251	211	182	161	145	132	109	94	84	77	67	62	55	52	46
250	299	241	202	175	154	138	126	104	90	80	73	64	58	52	49	42
260	288	231	194	167	148	133	121	99	85	76	69	60	55	48	45	39
270	277	222	186	161	142	127	116	95	82	72	66	57	52	46	42	36
280	267	214	180	155	137	122	111	91	78	69	63	54	49	43	40	34
290	257	207	173	149	132	118	107	88	75	66	60	52	46	41	38	32
300	249	200	167	144	127	114	103	84	72	64	57	49	44	38	35	29

2) *For vertically stiffened webs with horizontal stiffeners* — as described in **6.7.4.3**, the clear distance between the tension flanges (angles, flange plate or tongue plate) and the horizontal stiffener.

t = the thickness of the web.

NOTE 1 — For the minimum thickness of web plates and the design of web stiffeners, *see* **6.7.3** and **6.7.4**.

NOTE 2 — The allowable stresses given in the Tables 6.6A, 6.6B and 6.6C apply provided any reduction of the web cross section is due only to rivet holes, etc. Where large apertures are cut in the web, a special analysis shall be made to ensure that the maximum permissible average shear stresses laid down in this standard are not exceeded.

NOTE 3 — Compliance with this subclause shall be deemed to satisfy the requirements of **6.4.1**.

6.4.2.1 The cross sections of the web shall be taken as follows:

For rolled I-beams and channels	The depth of the beam multiplied by web thickness
For plate girders	The depth of the web plate multiplied by its thickness

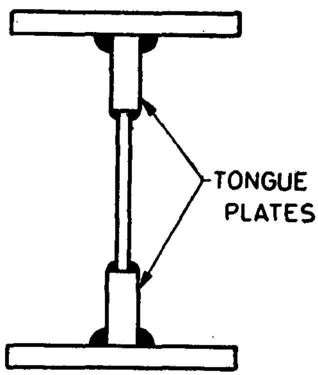


FIG. 6.1 TONGUE PLATES

TABLE 6.6A PERMISSIBLE AVERAGE SHEAR STRESS τ_{va} IN STIFFENED WEBS OF STEEL WITH f_y = 250 MPa

(*Clause* 6.4.2)

d/t	Stress τ_{va} (MPa) for Different Distances c Between Stiffeners												
	0·3d	0·4d	0·5d	0·6d	0·7d	0·8d	0·9d	1·0d	1·1d	1·2d	1·3d	1·4d	1·5d
90	100	100	100	100	100	100	100	100	100	100	100	100	100
95	100	100	100	100	100	100	100	100	100	100	100	100	99
100	100	100	100	100	100	100	100	100	100	100	99	99	98
105	100	100	100	100	100	100	100	100	100	99	98	97	96
110	100	100	100	100	100	100	100	100	99	98	96	95	94
115	100	100	100	100	100	100	100	100	98	96	95	94	93
120	100	100	100	100	100	100	100	98	96	95	93	92	91
125	100	100	100	100	100	100	98	97	95	93	92	91	90
130	100	100	100	100	100	99	97	96	94	92	90	89	88
135	100	100	100	100	100	98	96	94	92	90	89	87	86
140	100	100	100	100	99	96	95	93	91	89	87	86	85
150	100	100	100	100	97	94	92	90	88	86	84	83	81
160	100	100	100	98	94	92	89	88	85	83	81	80	78
170	100	100	100	96	92	89	87	85	82	80	78	76	75
180	100	100	98	94	90	87	84	82	80	77	75	73	72
190	100	100	97	92	88	84	82						
200	100	100	95	90	86	82	81						
210	100	99	93	88	83	81							
220	100	98	91	86	81	80							
230	100	96	90	84	79								
240	100	95	88	83	77								
250	100	93	86	82	74								
260	100	92	85	81									
270	99	90	84	81									

Non-applicable zone.

Note — Intermediate values may be obtained by linear interpolation.

TABLE 6.6 B PERMISSIBLE AVERAGE SHEAR τ_{va} IN STIFFENED WEBS OF STEEL WITH f_y = 340 MPa

(*Clause* 6.4.2)

d/t	Stress τ_{va} (MPa) for Different Distances *c* Between Stiffeners												
	0·3*d*	0·4*d*	0·5*d*	0·6*d*	0·7*d*	0·8*d*	0·9*d*	1·0*d*	1·1*d*	1·2*d*	1·3*d*	1·4*d*	1·5*d*
75	136	136	136	136	136	136	136	136	136	136	136	136	136
80	136	136	136	136	136	136	136	136	136	136	136	136	136
85	136	136	136	136	136	136	136	136	136	136	136	134	133
90	136	136	136	136	136	136	136	136	136	135	133	132	131
95	136	136	136	136	136	136	136	136	135	133	131	129	128
100	136	136	136	136	136	136	136	135	132	130	128	127	126
105	136	136	136	136	136	136	135	133	130	128	126	124	123
110	136	136	136	136	136	135	133	131	128	126	124	122	120
115	136	136	136	136	136	133	131	129	126	123	121	119	118
120	136	136	136	136	135	131	129	127	124	121	119	117	115
125	136	136	136	136	133	129	127	125	121	119	116	114	113
130	136	136	136	135	131	127	125	122	119	116	114	112	110
135	136	136	136	134	129	126	123	120	117	114	111	109	108
140	136	136	136	132	127	124	121	118	115	112	109	107	105
150	136	136	135	129	124	120	117	114	110	107	104	102	100
160	136	136	132	126	120	116	113	110	106	102	99	97	95
170	136	136	129	123	117	112	109	106	101	98	95	92	90
180	136	135	127	119	113	108	105	102	97	93	90	87	84
190	136	133	124	116	110	105	100						
200	136	130	121	113	106	101	96						
210	136	128	118	110	103	97							
220	136	126	116	107	99	93							
230	135	123	113	103	96								
240	134	121	110	100	92								
250	132	119	107	97	89								
260	130	116	104	94									
270	128	114	102	91									

Non-applicable zone.

NOTE — Intermediate values may be obtained by linear interpolation.

TABLE 6.6 C PERMISSIBLE AVERAGE SHEAR STRESS τ_{va} IN STIFFENED WEBS OF STEEL WITH $f_y = 400$ MPa

(*Clause* 6.4.2)

d/t	Stress τ_{va} (MPa) for Different Distances c Between Stiffeners												
	0·3d	0·4d	0·5d	0·6d	0·7d	0·8d	0·9d	1·0d	1·1d	1·2d	1·3d	1·4d	1·5d
70	160	160	160	160	160	160	160	160	160	160	160	160	160
75	160	160	160	160	160	160	160	160	160	160	160	160	159
80	160	160	160	160	160	160	160	160	160	160	159	157	156
85	160	160	160	160	160	160	160	160	160	158	156	154	152
90	160	160	160	160	160	160	160	160	157	155	152	151	149
95	160	160	160	160	160	160	159	157	154	152	149	147	146
100	160	160	160	160	160	160	157	155	151	149	146	144	143
105	160	160	160	160	160	157	154	152	149	146	143	141	139
110	160	160	160	160	159	155	152	149	146	143	140	138	136
115	160	160	160	160	156	152	149	147	143	140	137	135	133
120	160	160	160	159	154	150	147	144	140	137	134	132	129
125	160	160	160	157	152	147	144	141	137	134	131	128	126
130	160	160	160	155	150	145	141	139	134	131	128	125	123
135	160	160	160	153	147	143	139	136	132	128	125	122	120
140	160	160	158	151	145	140	136	133	129	125	122	119	116
150	160	160	155	147	141	135	131	128	123	119	115	112	110
160	160	160	151	143	136	130	126	123	117	113	109	106	103
170	160	158	148	139	132	126	121	117	112	107	103	100	97
180	160	155	144	135	127	121	116	112	106	101	97	93	90
190	160	152	140	131	123	116	111						
200	160	149	137	127	118	111	106						
210	160	146	133	123	114	106							
220	157	143	130	119	109	101							
230	155	140	126	114	105								
240	153	137	123	110	100								
250	151	134	119	106	96								
260	148	131	116	102									
270	146	128	112	98									

Non-applicable zone.

NOTE — Intermediate values may be obtained by linear interpolation.

6.5 Effective Span of Beams — The effective span of a beam shall be taken as the length between the centres of the supports, except in cases where the point of application of the reaction is taken as eccentricity to the support, when it shall be permissible to take the effective span as the length between the assumed points of application of reaction.

6.6 Effective Length of Compression Flanges

6.6.1 For simply supported beams and girders where no lateral restraint of the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length 'l' of the compression flanges to be used in **6.2** shall be taken as follows:

a) With ends of compression flanges unrestrained against lateral bending (that is, free to rotate in plan at the bearings)	l = span
b) With ends of compression flanges partially restrained against lateral bending (that is, not free to rotate in plan at the bearings)	$l = 0{\cdot}85 \times$ span
c) With ends of compression flanges fully restrained against lateral bending (that is, not free to rotate in plan at the bearings)	$l = 0{\cdot}7 \times$ span

Restraint against torsion can be provided by:

i) web or flange cleats, or

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

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

iv) their being built in to 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 flange are free to move laterally, the above values of the effective length shall be increased by 20 percent.

NOTE — The end restraint element shall be capable of safely resisting, in addition to wind and other applied external forces, a horizontal force acting at the bearing in a direction normal to the compression flange of the beam at the level of the centroid of the flange and having a value equal to not less than 2·5 percent of the maximum force occurring in the flange.

6.6.2 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 **6.6.1** the effective length of the compression flange shall be taken as the maximum distance, centre-to-centre, of the restraint members.

6.6.3 For cantilever beams of projecting length 'L' the effective length 'l' to be used in **6.2** shall be taken as follows:

a) Built-in at the support, free at the end	$l = 0{\cdot}85\ L$
b) Built-in at the support, restrained against torsion at the end by continuous construction (*see* Fig. 6.2A)	$l = 0{\cdot}75\ L$
c) Built-in at the support, restrained against lateral deflection and torsion at the free end (*see* Fig. 6.2B)	$l = 0{\cdot}5\ L$
d) Continuous at the support, unrestrained against torsion at the support and free at the end (*see* Fig. 6.2C)	$l = 3\ L$
e) Continuous at the support with partial restraint against torsion of the support and free at the end (*see* Fig. 6.2D)	$l = 2\ L$
f) Continuous at the support, restrained against torsion at the support and free at the end (*see* Fig. 6.2E)	$l = L$

L = length of cantilever

If there is a degree of fixity at the free end, the effective length shall be multiplied by

$\frac{0{\cdot}5}{0{\cdot}85}$ in (b) and (c) above, and by $\frac{0{\cdot}75}{0{\cdot}85}$ in (d), (e) and (f) above.

6.6.4 Where beams support slab construction, the beam shall be deemed to be effectively restrained laterally if the frictional or positive connection of the slab to the beam is capable of resisting a lateral force of 2·5 percent of the maximum force in the compression flange of the beam, considered as distributed uniformly along the flange. Furthermore, the slab construction shall be capable of resisting this lateral force in flexure and shear.

6.6.5 For beams which are provided with members giving effective lateral restraint of the compression flange at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2·5 percent of the maximum force in the compression flange taken as divided equally between the number of points at which the restraint members occur.

6.6.6 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2 percent of the maximum flange force in one beam only.

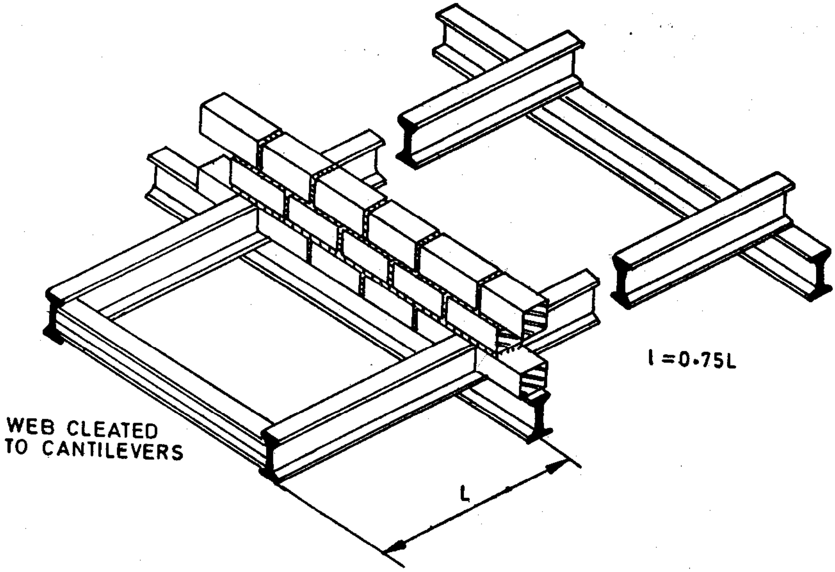


Fig. 6.2A Cantilever Built-in at Support, Restrained Against Torsion at the End

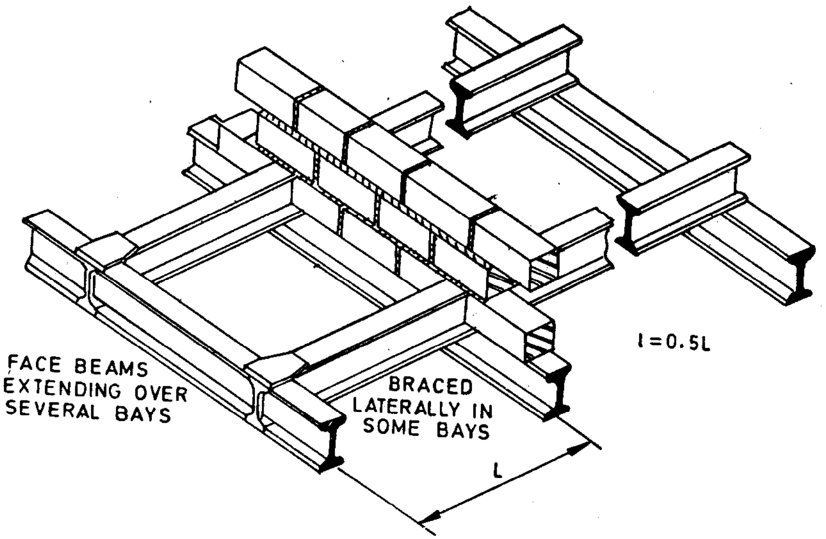


Fig. 6.2B Cantilever Built-in at Support, Restrained Laterally at the End

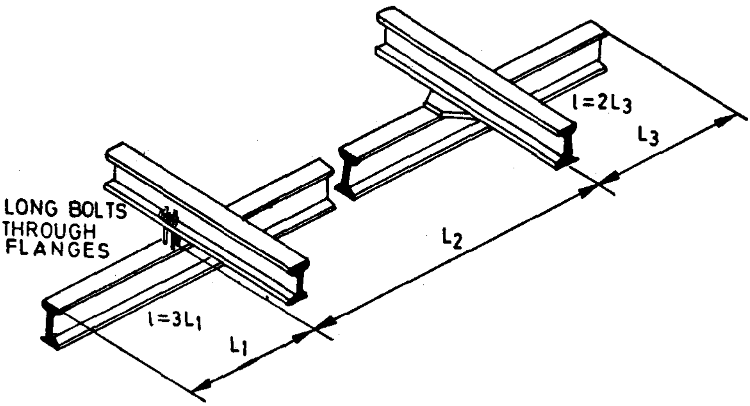


Fig. 6.2C Cantilever L_1 Continuous at the Support, Unrestrained Against Torsion at the Support and Unrestrained at the End

Fig. 6.2D Cantilever L_3 Continuous at the Support, Partially Restrained Against Torsion at the Support and Unrestrained at the End

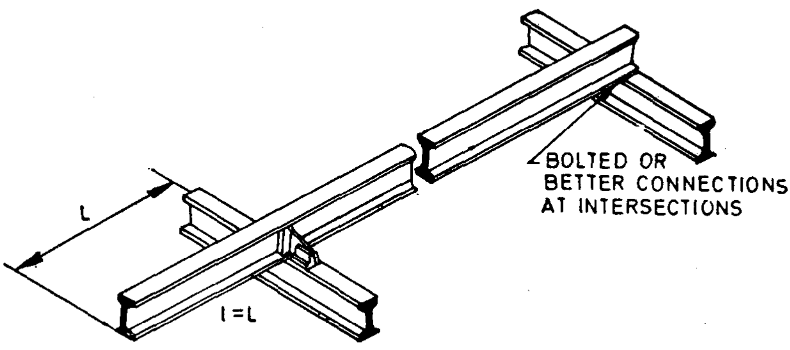


Fig. 6.2E Cantilever Span Continuous at the Support, Fully Restrained Against Torsion at the Support and Unrestrained at the Free End

6.6.6.1 In the case of a series of latticed beams, girders or roof trusses which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2·5 percent of the maximum force in the compression flange plus 1·25 percent of this force for every member of the series other than the first up to a maximum total of 7·5 percent.

6.7 Design of Beams and Plate Girders with Solid Webs

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

6.7.1.1 The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members (*see* **3.5.2.1** and **3.5.2.2**) 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 (*see* **3.6**).

The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in **3.5.2.1** and **3.6** of this Code.

The effective sectional area for parts in shear shall be taken as specified in **6.7.3.4**.

6.7.2 *Flanges*

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

a) In exposed situations where flange plates are used, at least one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is machined flush with the flange angles. Where two or more flange plates are used on the one flange, tacking rivets shall be provided, if necessary, to comply with the requirements of **8.10.2** and **8.10.3**.

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

c) The outstand of flange plates, that is the projection beyond the outer line of connections to flange angles, channel or joist flanges, or, in the case of welded constructions, their projection beyond the face of the web or tongue plate, shall not exceed the values given in **3.5.2**.

d) In the case of box girders, the thickness of any plate, or the aggregate thickness of two or more plates when these plates are tacked together to form the flange, shall satisfy the requirements given in **3.5.2**.

6.7.2.2 *Flange splices* — Flange joints preferably should not be located at points of maximum stress. Where splice plates are used, their area shall be not less than 5 percent in excess of the area of the flange element spliced; their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough rivets or welds on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. In welded construction, flange plates shall be joined by complete penetration butt welds, wherever possible. These butt welds shall develop the full strength of the plates.

6.7.2.3 *Connection of flanges to web* — The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the maximum horizontal shear force resulting from the bending moments in the girder, combined with any vertical loads which are directly applied to the flange.

6.7.2.4 *Dispersion of load through flange to web* — Where a load is directly applied to a top flange, it shall be considered as dispersed uniformly at an angle of 30 degrees to the horizontal.

6.7.3 *Web Plates*

6.7.3.1 *Minimum thickness* — The thickness of the web plate shall be not less than the following:

a) For unstiffened webs: the greater of

$$\frac{d_1 \sqrt{\tau_{va,\,cal}}}{816} \text{ and } \frac{d_1 \sqrt{f_y}}{1\,344} \text{ but not less than } \frac{d_1}{85}$$

where

d_1 = depth of web as defined in **1.3**, and

$\tau_{va,\,cal}$ = calculated average stress in the web due to shear force.

b) For vertically stiffened webs: the greater of

1/180 of the smallest clear panel dimension

and $\dfrac{d_2\sqrt{f_y}}{3\,200}$ but not less than $\dfrac{d_2}{200}$

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: the greater of

1/180 of the smaller dimension in each panel,

and $\dfrac{d_2\sqrt{f_y}}{4\,000}$ but not less than $\dfrac{d_2}{250}$

d) When there is also a horizontal stiffener at the neutral axis of the girder: the greater of

1/180 of the smaller dimension in each panel,

and $\dfrac{d_2\sqrt{f_y}}{6\,400}$ but not less than $\dfrac{d_2}{400}$

In (b), (c) and (d) above, d_2 is twice the clear distance from the compression flange angles, or plate, or tongue plate to the neutral axis.

In the case of welded crane gantry plate girders intended for carrying cranes with a lifting load of 15 tonnes or more, the thickness of web plate shall be not less than 8 mm.

The minimum thickness of web plates for different yield stress values are given in Table 6.7 for information.

NOTE — In no case shall the greater clear dimension of a web panel exceed 270 t, nor the lesser clear dimension of the same panel exceed 180 t, where t is the thickness of the web plate.

6.7.3.2 *Riveted construction* — For girders in exposed situations and which do not have flange plates for their entire length, the top edge of the web plate shall be flush with or above the angles, as specified by the engineer, and the bottom edge of the web plate shall be flush with or set back from the angles, as specified by the engineer.

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

TABLE 6.7 MINIMUM THICKNESS OF WEB

(*Clause* 6.7.3.1)

	Minimum Thickness of Web for Yield Stress f_y (in MPa) of																
f_y	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
$\frac{d_1\sqrt{f_y}}{1\,344}$	$\frac{d_1}{85}$	$\frac{d_1}{85}$	$\frac{d_1}{85}$	$\frac{d_1}{85}$	$\frac{d_1}{83}$	$\frac{d_1}{80}$	$\frac{d_1}{78}$	$\frac{d_1}{75}$	$\frac{d_1}{73}$	$\frac{d_1}{71}$	$\frac{d_1}{69}$	$\frac{d_1}{67}$	$\frac{d_1}{66}$	$\frac{d_1}{63}$	$\frac{d_1}{61}$	$\frac{d_1}{60}$	$\frac{d_1}{58}$
$\frac{d_2\sqrt{f_y}}{3\,200}$	$\frac{d_2}{200}$	$\frac{d_2}{200}$	$\frac{d_2}{200}$	$\frac{d_2}{200}$	$\frac{d_2}{198}$	$\frac{d_2}{191}$	$\frac{d_2}{185}$	$\frac{d_2}{179}$	$\frac{d_2}{174}$	$\frac{d_2}{169}$	$\frac{d_2}{164}$	$\frac{d_2}{160}$	$\frac{d_2}{156}$	$\frac{d_2}{151}$	$\frac{d_2}{146}$	$\frac{d_2}{142}$	$\frac{d_2}{138}$
$\frac{d_2\sqrt{f_y}}{4\,000}$	$\frac{d_2}{250}$	$\frac{d_2}{250}$	$\frac{d_2}{250}$	$\frac{d_2}{250}$	$\frac{d_2}{248}$	$\frac{d_2}{239}$	$\frac{d_2}{231}$	$\frac{d_2}{224}$	$\frac{d_2}{217}$	$\frac{d_2}{211}$	$\frac{d_2}{205}$	$\frac{d_2}{200}$	$\frac{d_2}{195}$	$\frac{d_2}{189}$	$\frac{d_2}{183}$	$\frac{d_2}{177}$	$\frac{d_2}{172}$
$\frac{d_2\sqrt{f_y}}{6\,400}$	$\frac{d_2}{400}$	$\frac{d_2}{400}$	$\frac{d_2}{400}$	$\frac{d_2}{400}$	$\frac{d_2}{396}$	$\frac{d_2}{382}$	$\frac{d_2}{370}$	$\frac{d_2}{358}$	$\frac{d_2}{348}$	$\frac{d_2}{338}$	$\frac{d_2}{328}$	$\frac{d_2}{320}$	$\frac{d_2}{312}$	$\frac{d_2}{302}$	$\frac{d_2}{292}$	$\frac{d_2}{284}$	$\frac{d_2}{276}$

6.7.3.4 *Effective sectional area*

a) *Web of plate girder* — The effective cross-sectional area shall be taken as the full depth of the web plate multiplied by the thickness.

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

b) *Rolled beams and channels* — The effective cross-sectional area for shear shall be taken as the full depth of the beam or channel multiplied by its web thickness. For other sections the maximum shear stress shall be computed from the whole area of the cross section, having regard to the actual distribution of shear stress.

c) Webs which have openings larger than those normally used for rivets or other fastenings require special analysis to ensure that the permissible stress as specified in this standard are not exceeded.

6.7.3.5 *Splices in webs* — Splices in the webs of the plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In riveted construction, splice plates shall be provided on each side of the web. In welded construction, web splices shall preferably be made with complete penetration butt welds.

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

6.7.4 *Intermediate Web Stiffeners for Plate Girders*

6.7.4.1 *General* — When the thickness of the web is less than the limits specified in **6.7.3.1** (a) vertical stiffeners shall be provided throughout the length of the girder. When the thickness of the web is less than the limits specified in **6.7.3.1** (b) horizontal stiffeners shall be provided in addition to the vertical stiffeners.

In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t, where t is the thickness of the web plate.

6.7.4.2 *Vertical stiffeners* — Where vertical stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than 1·5 d and not less than 0·33 d, where d is the depth as defined in **6.4.2** (definition 1). Where horizontal stiffeners are provided d in mm shall be taken as the clear distance between the horizontal stiffener and the tension flange (farthest flange) ignoring fillets. These vertical stiffeners shall be designed so that I is not less than

$$1\cdot5 \times \frac{d^3 \times t^3}{c^2}$$

where

I = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web,

t = the minimum required thickness of the web, and

c = the maximum permitted clear distance between vertical stiffener for thickness t.

NOTE — If the thickness of the web is made greater, or the spacing of stiffeners made smaller than that required by the standard, the moment of inertia of the stiffener 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. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange, but need not have the ends fitted to provide a tight bearing on the flange.

6.7.4.3 *Horizontal stiffeners* — Where horizontal stiffeners are used in addition to vertical stiffeners, they shall be as follows:

a) One horizontal stiffener shall be placed on the web at a distance from the compression flange equal to 2/5 of the distance from the compression flange to the neutral axis when the thickness of the web is less than the limits specified in **6.7.3.1** (b). This stiffener shall be designed so that I is not less than $4c.t^3$ where I and t are as defined in **6.7.4.2** and c is the actual distance between the vertical stiffeners;

b) A second horizontal stiffener (single or double) shall be placed at the neutral axis of the girder when the thickness of the web is less than the limit specified in **6.7.3.1** (c). This stiffener shall be designed so that I is not less than $d_2.t^3$ where d_2 also in mm, I and t are as defined in **6.7.4.2** and d_2 is as defined in **6.7.3.1**;

c) Horizontal web stiffeners shall extend between vertical stiffeners but need not be continuous over them; and

d) Horizontal stiffeners may be in pairs arranged on each side of the web, or single.

6.7.4.4 *Outstand of stiffeners* — Unless the outer edge of each stiffener is continuously stiffened, the outstand of all stiffeners from the web shall be not more than $\frac{256.t}{\sqrt{f_y}}$ for sections and 12 t for flats where t is the thickness of the section or flat.

6.7.4.5 *External forces on intermediate stiffeners* — When vertical intermediate stiffeners are subjected to bending moments and shears due to eccentricity of vertical loads, or the action of transverse forces, the moment of inertia of the stiffeners given in **6.7.4.2** shall be increased as shown below:

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

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

b) Lateral loading on stiffener:

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

where

M = the applied bending moment, KNm;

D = overall depth of girder, in mm;

E = Young's modulus, 2×10^5 MPa;

t = thickness of web, mm; and

V = the transverse force in KN to be taken by the stiffener and deemed to be applied at the compression flange of the girder.

6.7.4.6 *Connections of intermediate stiffeners to web* — Intermediate vertical and horizontal stiffeners not subjected to external loads shall be connected to the web by rivets or welds, so as to withstand a shearing force, between each component of the stiffener and the web of not less than

$$\frac{125t^2}{h}\ \text{kN/m}$$

where

t = the web thickness in mm, and

h = the outstand of stiffener in mm.

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

6.7.5 *Load Bearing Web Stiffeners*

6.7.5.1 *All sections* — For any section, load bearing stiffeners shall be provided at points of concentrated load (including points of support) where the concentrated load or reaction exceeds the value of

$$\sigma_{ac}.t.B$$

where

σ_{ac} = the maximum permissible axial stress for columns as given under **5.1** for a slenderness ratio $\frac{d_1}{t}\sqrt{3}$;

t = web thickness;

B = the length of the stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of the neutral axis, plus the thickness of the scating angle, if any. The stiff portion of a bearing is that length which cannot deform appreciably in bending and shall not be taken as greater than half the depth of beam for simply supported beams and the full depth of the beams continuous over a bearing; and

d_1 = clear depth of web between root fillets.

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

6.7.5.2 *Plate girders* — In addition to the requirements of **6.7.5.1**, load bearing stiffness shall be provided also at the supports where either:

a) the web is overstressed in shear [*see* **6.7.3.1** (a)], or

b) the web is otherwise overstressed at support or at the web connection.

6.7.5.3 *Design of load bearing stiffeners*

a) Load bearing stiffeners shall be designed as columns 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 and equal, where available, to 20 times the web thickness. The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the working stress shall be in accordance with the appropriate allowable value for a compression member assuming an effective length equal to 0·7 of the length of the stiffeners;

b) The outstanding legs of each pair of stiffeners shall be so proportioned that the bearing stress on that part of their area clear of the root of the flange or flange angles or clear of the welds does not exceed the bearing stress specified in **6.3**;

c) Stiffeners shall be symmetrical about the web, where possible and at points of support shall project as nearly as practicable to the outer edges of the flanges;

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

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

f) Bearing stiffeners shall not be joggled and shall be solidly packed throughout; and

g) For plate girders, where load bearing stiffeners at supports are the sole means of providing restraint against torsion (*see* **6.6.4**) the moment of inertia, I, of the stiffener about the centre line of the web plate, shall be not less than

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

where

D = overall depth of the girder,

T = maximum thickness of compression flange,

R = reaction of the beam at the support, and

W = total load on the girder between supports.

In addition, the bases of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to the horizontal force specified in the Note under **6.6.1**.

6.8 Box Girders — The design and detailing of box girders shall be such as to give full advantage of its higher load carrying capacity. The diaphragms and horizontal stiffeners should conform to **6.7.3** and **6.7.4**.

6.8.1 All diaphragms shall be connected such as to transfer the resultant shears to the web and flanges.

6.8.2 Where the concentrated or moving load does not come directly on top of the web, the local effect shall be considered for the design of flanges and the diaphragms.

6.9 Purlins

6.9.1 All purlins shall be designed in accordance with the requirements for uncased beams (*see* **6.2.1** and Table 3.1), and the limitations of bending stress based on lateral instability of the compression flange and the limiting deflection specified under **3.13** may be waived for the design of purlins. The maximum fibre stress shall not exceed the values specified in **6.2.1** except as provided under **3.9** for increase of stress. The calculated deflections should not exceed those permitted for the type of roof cladding used. In calculating the bending moment advantage may be taken of the continuity of the purlin over supports. The bending stresses about the two axes should be determined separately and checked in accordance with **7.1.1**. Open web purlins shall be designed as trusses.

6.9.2 *Angle purlins of steel conforming to Grades* Fe 410-0. F*e* 410-*S or* F*e* 410-*W and slopes not exceeding* 30° *Pitch* — As an alternate to the general design procedure given in **6.9.1** angle purlins of roofs with slopes not exceeding 30 degrees may be designed, if the following requirements which are based on a minimum imposed load of 0·75 kN/m² are fulfilled:

a) The width of leg or the depth of the purlin in the plane appropriate to the incidence of the maximum load or maximum component of the load is not less than $L/45$;

b) The width of the other leg or width of the purlin is not less than $L/60$;

c) The maximum bending moment in a purlin may be taken as $\frac{WL}{10}$ where W is the total distributed load on the purlin including wind load. The loads shall be assumed as acting normal to the roof in which case the bending about the minor axis may be neglected. L shall be taken as distance centre-to-centre of the rafters or other supports of the purlins; and

d) Under the bending moment calculated as in (c) above, the maximum fibre stress shall not exceed the appropriate value of σ_{bc} or σ_{bt} given in **6.2** except as provided under **3.9** for increase of stresses. The calculated deflection should not exceed those permitted for the type of cladding used.

6.10 Side and End Sheeting Rails — Side and end sheeting rails shall be designed for wind pressures and vertical loads, if any; and the requirements of, as regards limiting deflection and lateral stability of beams, the same provisions as given in **6.9.1** shall apply.

SECTION 7 COMBINED STRESSES

7.1 Combination of Direct Stresses

7.1.1 *Combined Axial Compression and Bending* — Members subjected to axial compression and bending shall be proportioned to satisfy the following requirements:

a) $$\frac{\sigma_{ac,\ cal.}}{\sigma_{ac}} + \frac{C_{mx} \cdot \sigma_{bcx,\ cal.}}{\left\{1 - \frac{\sigma_{ac,\ cal.}}{0.60 f_{ccx}}\right\}\sigma_{bcx}} + \frac{C_{my} \cdot \sigma_{bcy,\ cal.}}{\left\{1 - \frac{\sigma_{ac,\ cal.}}{0.60 f_{ccy}}\right\}\sigma_{bcy}} \leqslant 1.0$$

However, if the ratio $\frac{\sigma_{ac,\ cal.}}{\sigma_{ac}}$ is less than 0.15, the following expression may be used in lieu of the above:

$$\frac{\sigma_{ac,\ cal.}}{\sigma_{ac}} + \frac{\sigma_{bcx,\ cal.}}{\sigma_{bcx}} + \frac{\sigma_{bcy,\ cal.}}{\sigma_{bcy}} \leqslant 1.0$$

The value of σ_{bcx} and σ_{bcy} to be used in the above formulae shall each be lesser of the values of the maximum permissible stresses σ_{bc} given in Section 6 for bending about the appropriate axis.

b) At a support and using the values σ_{bcx} and σ_{bcy} at the support:

$$\frac{\sigma_{ac,\ cal}}{0.60 f_y} + \frac{\sigma_{bcx,\ cal.}}{\sigma_{bcx}} + \frac{\sigma_{bcy,\ cal.}}{\sigma_{bcy}} \leqslant 1.0$$

For an encased strut where an allowance is made for the force carried by the concrete in accordance with **10.1.1** the ratio of $\frac{\sigma_{ac,\ cal.}}{\sigma_{ac}}$ shall be replaced by the ratio of the calculated axial force on the strut to the maximum permissible axial force determined as per **10.1.2.**

7.1.2 *Combined Axial Tension and Bending* — A member subjected to both axial tension and bending shall be proportioned so that the following condition is satisfied:

$$\frac{\sigma_{at,\ cal.}}{0.60 f_y} + \frac{\sigma_{btx,\ cal.}}{0.66 f_y} + \frac{\sigma_{bty,\ cal.}}{0.66 f_y} \leqslant 1$$

7.1.3 *Symbols* — The symbols used in **7.1.1** and **7.1.2** shall have the following meaning:

$\sigma_{ac,\ cal.}$ = calculated average axial compressive stress

$\sigma_{at,\ cal.}$ = calculated average axial tensile stress

$\sigma_{bc,\ cal.}$ = calculated bending compressive stress in extreme fibre

$\sigma_{bt,\ cal.}$ = calculated bending tensile stress in extreme fibre

σ_{ac} = permissible axial compressive stress in the member subject to axial compressive load only

σ_{at} = permissible axial tensile stress in the member subject to axial tensile load only

σ_{bc} = permissible bending compressive stress in extreme fibre

σ_{bt} = permissible bending tensile stress in extreme fibre

f_{cc} = elastic critical stress in compression $= \frac{\pi^2 E}{\lambda^2}$

$\lambda \left(= \frac{l}{r} \right)$ = slenderness ratio in the plane of bending

x, y = represent x-x and y-y planes

C_m = a coefficient whose value shall be taken as follows:

a) For member in frames where side sway is not prevented:

$$C_m = 0{\cdot}85$$

b) For members in frames where side sway is prevented and not subject to transverse loading between their supports in the plane of bending:

$$C_m = 0{\cdot}6 - 0{\cdot}4\,\beta \geqslant 0{\cdot}4$$

NOTE 1 — β is the ratio of smaller to the larger moments at the ends of that portion of the unbraced member in the plane of bending under consideration.

NOTE 2 — β is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

c) For members in frames where side sway 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 analysis, the following values may be used:

For members whose ends are restrained against rotation

$$C_m = 0{\cdot}85$$

For members whose ends are unrestrained against rotation

$$C_m = 1{\cdot}00$$

7.1.4 *Bending and Shear* — Irrespective of any increase in the permissible stress specified in **3.9**, the equivalent stress $\sigma_{e,\,cal.}$, due to co-existent bending (tension or compression) and shear stresses obtained from the formula given in **7.1.4.1** shall not exceed the value:

$$\sigma_e = 0{\cdot}9 f_y$$

where

σ_e = maximum permissible equivalent stress.

7.1.4.1 The equivalent stress $\sigma_{e,\ cal.}$ is obtained from the following formula:

$$\sigma_{e,\ cal.} = \sqrt{\sigma_{bt}^2{}_{,\ cal.} + 3\tau_{vm}^2{}_{,\ cal.}} \quad \text{or}$$
$$= \sqrt{\sigma_{bc}^2{}_{,\ cal.} + 3\tau_{vm}^2{}_{,\ cal.}}$$

7.1.5 *Combined, Bearing, Bending and Shear Stresses* — Where a bearing stress is combined with tensile or compressive, bending and shear stresses under the most unfavourable condition of loading, the equivalent stress $\sigma_{e,\ cal.}$ obtained from the following formulae, shall not exceed $\sigma_e = 0{\cdot}9 f_y$.

$$\sigma_{e,\ cal.} = \sqrt{\sigma_{bt}^2{}_{,\ cal.} + \sigma_p^2{}_{.\ cal.} + \sigma_{bt,\ cal.}\,\sigma_{p,\ cal.} + 3\,\tau_{vm}^2{}_{,\ cal.}}$$

or

$$\sigma_{e,\ cal.} = \sqrt{\sigma_{bc}^2{}_{,\ cal.} + \sigma_p^2{}_{,\ cal.} + \sigma_{bc,\ cal.}\,\sigma_{p,\ cal.} + 3\tau_{vm}^2{}_{,\ cal.}}$$

7.1.6 In **7.1.4** and **7.1.5** $\sigma_{bt,\ cal.}$; $\sigma_{bc,\ cal.}$; $\tau_{vm,\ cal.}$ and $\sigma_{p,\ cal.}$ are the numerical values of the co-existent bending (compression or tension), shear and bearing stresses. When bending occurs about both axes of the member, $\sigma_{bt.\ cal}$ and $\sigma_{bc,\ cal.}$ shall be taken as the sum of the two calculated fibre stresses. σ_e is the maximum permissible equivalent stress.

SECTION 8 CONNECTIONS

8.0 General — As much of the work of fabrication as is reasonably practicable shall be completed in the shops where the steel work is fabricated.

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

8.2 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 rivets and turned and fitted bolts may be considered as acting together to share the load. In all other connections sufficient number of one type of fastening shall be provided to communicate the entire load for which the connection is designed.

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

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

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

8.4 Bearing Brackets — Wherever practicable, connections of beams to columns shall include a bottom bracket and top cleat. Where web cleats are not provided, the bottom bracket shall be capable of carrying the whole of the load.

8.5 Gussets — Gusset plates shall be designed to resist the shear, direct and flexural stresses acting on the weakest or critical section. Re-entrant cuts shall be avoided as far as practicable.

8.6 Packings

8.6.1 *Rivets or Bolts Through Packings* — Number of rivets or bolts carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2·5 percent for each 2·0 mm thickness of packing except that, for packings having a thickness of 6 mm or less, no increase need be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing. The additional rivets or bolts should preferably be placed in an extension of the packing.

8.6.2 *Packings in Welded Construction* — Where a packing is used between two parts, the packing and the welds connecting it to each part shall be capable of transmitting the load between the parts. Where the packing is too thin to carry the load or permit the provision of adequate welds, the load shall be transmitted through the welds alone, the welds being increased in size by an amount equal to the thickness of the packing.

8.6.3 *Packing Subjected to Direct Compression only* — Where properly fitted packings are subjected to direct compression only, the provisions under **8.6.1** and **8.6.2** shall not apply.

8.7 Separators and Diaphragms — Where two or more rolled steel joists or channels are used side by side to form a girder, they shall be connected together at intervals of not more than 1 500 mm except in the case of grillage beams encased in concrete, where suitable provision shall be made to maintain correct spacing. Bolts and separators may be used provided that in beams having a depth of 300 mm or more, not fewer than 2 bolts are used with each separator. When loads are required to be carried from one beam to the other or are required to be distributed between the beams, diaphragms shall be used, designed with sufficient stiffness to distribute the load.

8.8 Lug Angles

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

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

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

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

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

8.8.6 Where lug angles are used to connect an angle member the whole area of the member shall be taken as effective notwithstanding the requirements of Section 3 and Section 5 of this code.

8.9 Permissible Stresses in Rivets and Bolts

8.9.1 *Calculation of Stresses* — In calculating shear and bearing stresses the effective diameter of a rivet shall be taken as the hole diameter and that of a bolt as its nominal diameter. In calculating the axial tensile stress in a rivet the gross area shall be used and in calculating the axial tensile stress in a bolt or screwed tension rod the net area shall be used.

8.9.2 *Gross and Net Areas of Rivets and Bolts*

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

8.9.2.2 The net sectional area of a bolt or screwed tension rod shall be taken as the area of the root of the threaded part or cross-sectional area of the unthreaded part whichever is lesser.

NOTE — The net sectional areas of bolts are given in IS : 1364-1967 and IS : 1367-1967.

8.9.3 *Area of Rivet and Bolt Holes* — The diameter of a rivet hole shall be taken as the nominal diameter of a rivet plus 1·5 mm for rivets of nominal diameter less than or equal to 25 mm, and 2·0 mm for rivets of nominal diameter exceeding 25 mm, unless otherwise specified. The diameter of a bolt hole shall be taken as the nominal diameter of the bolt plus 1·5 mm unless specified otherwise.

8.9.4 *Stresses in Rivets, Bolts and Welds*

8.9.4.1 The calculated stress in a mild steel shop rivet or in a bolt of property class 4.6 (*see* IS : 1367-1967) shall not exceed the values given in Table 8.1.

TABLE 8.1 MAXIMUM PERMISSIBLE STRESS IN RIVETS AND BOLTS

DESCRIPTION OF FASTENERS	AXIAL TENSION, σ_{tf},	SHEAR, τ_{vf}	BEARING, σ_{pf}
(1)	(2)	(3)	(4)
	MPa	MPa	MPa
Power-driven rivets	100	100	300
Hand-driven rivets	80	80	250
Close tolerance and turned bolts	120	100	300
Bolts in clearance holes	120	80	250

8.9.4.2 The permissible stress in a high tensile steel rivet shall be those given in Table 8.1 multiplied by the ratio of the tensile strength of the rivet material to the tensile strength as specified in IS : 1148-1982

NOTE — For field rivets the permissible stresses shall be reduced by 10 percent.

8.9.4.3 The permissible stress in a bolt (other than a high strength friction grip bolt) of property class higher than 4·6 shall be those given in Table 8.1 multiplied by the ratio of its yield stress or 0·2 percent proof stress or 0·7 times its tensile strength, whichever is the lesser, to 235 MPa.

8.9.4.4 The calculated bearing stress of a rivet or bolt on the parts connected by it shall not exceed : (a) the value f_y for hand driven rivets or bolts in clearance holes, and (b) the value 1·2 f_y for power driven rivets or close tolerance and turned bolts. f_y is the yield stress of the connected parts.

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

8.9.4.5 *Combined shear and tension* — Rivets and bolts subject to both shear and axial tension shall be so proportioned that the shear and axial stresses calculated in accordance with **8.9.1** do not exceed the respective allowable stresses τ_{vf} and σ_{tf} and the expression $\left\{ \frac{\tau_{vf,\,cal.}}{\tau_{vf}} + \frac{\sigma_{tf,\,cal.}}{\sigma_{tf}} \right\}$ does not exceed 1·4.

8.9.4.6 *High strength friction grip bolts* – The provisions contained in **8.9.4.1** to **8.9.4.5** do not apply to high strength friction grip bolts, which shall be used in conformity with IS : 4000-1967.

8.9.4.7 *Welds* — Permissible stress in welds shall be as specified in IS : 816-1969 and IS : 1323-1982.

8.10 Rivets and Riveting

8.10.1 *Pitch of Rivets*

a) *Minimum Pitch* — The distance between centres of rivets should be not less than 2·5 times the nominal diameter of the rivet.

b) *Maximum Pitch*

   i) The distance between centres of any two adjacent rivets (including tacking rivets) shall not exceed $32t$ or 300 mm, whichever is less, where t is the thickness of the thinner outside plate.

   ii) The distance between centres of two adjacent rivets, in a line lying in the direction of stress, shall not exceed $16t$ or 200 mm, whichever is less in tension members and $12t$ or 200 mm, whichever is less in compression members. In the case of compression members in which forces are transferred through butting faces, this distance shall not exceed 4·5 times the diameter of the rivets for a distance from the abutting faces equal to 1·5 times the width of the member.

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

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

8.10.2 *Edge Distance*

a) The minimum distance from the centre of any hole to the edge of a plate shall be not less than that given in Table 8.2.

b) Where two or more parts are connected together, a line of rivets or bolts shall be provided at a distance of not more than 37 mm + 4 t from the nearest edge, where t is the thickness in mm of the thinner outside plate. In the case of work not exposed to weather, this may be increased to 12 t.

TABLE 8.2 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
(1)	(2)	(3)
mm	mm	mm
13·5 and below	19	17
15·5	25	22
17·5	29	25
19·5	32	29
21·5	32	29
23·5	38	32
25·5	44	38
29·0	51	44
32·0	57	51
35·0	57	51

8.10.3 *Tacking Rivets* — In cases of members covered under **8.10.1**(b)(ii), when the maximum distance between centres of two adjacent rivets as specified in **8.10.1**(b)(ii) is exceeded, tacking rivets not subjected to calculated stress shall be used.

8.10.3.1 Tacking rivets shall have a pitch in line not exceeding 32 times the thickness of the outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the pitch in line shall not exceed 16 times, the thickness of the outside plate or 200 mm, whichever is less. In both cases, the lines of rivets shall not be apart at a distance greater than these pitches.

8.10.3.2 All the requirements specified in **8.10.3.1** shall apply to compression members generally, subject to the stipulation in this code affecting the design and construction of compression members.

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

8.10.3.4 For compression members covered in Section 5, the tacking rivets shall be at a pitch in line not exceeding 600 mm.

8.10.4 *Countersunk Heads* — For countersunk heads, one-half of the depth of the countersinking shall be neglected in calculating the length of the rivet in bearing. For rivets in tension with countersunk heads, the tensile value shall be reduced by 33·3 percent. No reduction need be made in shear.

8.10.5 *Long Grip Rivets* — Where the grip of rivets carrying calculated loads exceed 6 times the diameter of the holes, the number of rivets required by normal calculation shall be increased by not less than one percent for each additional 1·5 mm of grip; but the grip shall not exceed 8 times the diameter of the holes.

8.11 Bolts and Bolting

8.11.1 *Pitches, Edge Distances for Tacking Bolts* — The requirements for bolts shall be the same as for rivets given in **8.10** and its sub-clauses.

8.11.2 *Black Bolts* — The dimensions of black bolts shall conform to those given in IS : 1363-1967.

8.11.3 *Close Tolerance Bolts* — Close tolerance bolts shall conform to IS : 1364-1967.

8.11.4 *Turned Barrel Bolts* — The nominal diameter of the barrel shall be in multiples of 2 mm and shall be at least 2 mm larger in diameter than the screwed portion.

8.11.5 *Washers* — Washers with perfectly flat faces should be provided with all close tolerance bolts and turned barrel bolts. Steel or malleable cast iron tapered washers shall be provided for all heads and nuts bearing on bevelled surfaces.

8.11.6 *Locking of Nuts* — Wherever there is risk of the nuts becoming loose due to vibration or reversal of stresses, they shall be securely locked.

8.12 Welds and Welding — For requirements of welds and welding, reference shall be made to IS : 816-1969 and IS : 9595-1980.

SECTION 9 PLASTIC DESIGN

9.1 General

9.1.1 The structure or part of a structure may be proportioned on the basis of plastic design based on their maximum strength using the provisions contained in this section. Reference may also be made to SP (6) 6-1972.

9.1.2 The requirement of this standard regarding the maximum permissible stress shall be waived for this method. However, the design shall comply with all other requirements of this standard.

9.1.3 Members subjected to heavy impact and fatigue shall not be designed on the basis of plastic theory.

9.1.4 Steel conforming to Grade Fe 410-0 of IS : 1977-1975 shall not be used when the structure is designed on the basis of plastic theory.

9.2 Design

9.2.1 *Load Factors* — Structures or portions of structures proportioned using plastic design shall have sufficient strength as determined by plastic analysis to support the working loads multiplied by load factors as given below:

Working Loads	*Load Factor, Min*
Dead load	1·7
Dead load + imposed load	1·7
Dead load + load due to wind or seismic forces	1·7
Dead load + imposed load + load due to wind or seismic forces	1·3

9.2.2 *Deflection* — Deflections under working loads shall be in accordance with relevant provisions of this code.

9.2.3 *Beams*

9.2.3.1 The calculated maximum moment capacity, M_p, of a beam shall be

$$M_p = Z_p \cdot f_y$$

where

Z_p = plastic modulus of the section, and

f_y = yield stress of the material.

9.2.3.2 Plastic properties of Indian Standard medium weight beams are given in Appendix F for information.

9.2.4 *Tension Members* — The calculated maximum load capacity P_{at} of a tension member shall be $P_{at} = 0{\cdot}85\ A_s f_y$.

where

A_s = effective cross-sectional area of the member, and

f_y = yield stress of the steel.

9.2.5 *Struts* — The calculated maximum load capacity P_{ac} of a strut shall be

$$P_{ac} = 1{\cdot}7\ A_s\ \sigma ac$$

where σac is the maximum permissible stress in axial compression as given in **5.1** using an effective length l equal to the actual length L.

9.2.6 *Members Subjected to Combined Bending and Axial Forces (Beam-Column Members)*

9.2.6.1 The calculated maximum moment capacity M_{pc} of a member subjected to combined bending and axial forces, where P/P_y exceeds 0·15, shall be reduced below the value given in **9.2.3** and it shall satisfy the following requirements:

a) *Beams* — $\dfrac{P}{P_y} + \dfrac{M_{pc}}{1{\cdot}18\ M_p} \leqslant 1{\cdot}0$

b) *Slender struts* — A member where $\dfrac{P}{P_y}$ in addition to exceeding 0·15 also exceeds $\dfrac{1+\beta-\lambda_0}{1+\beta+\lambda_0}$ shall not be assumed to contain plastic hinges although it shall be permissible to design the member as an elastic part of a plastically designed structure. Such a member shall be designed according to the maximum permissible stress requirements satisfying:

$$\frac{P}{P_{ac}} + \frac{M_{pc}.C_m}{M_o\left(1 - \dfrac{P}{P_e}\right)} \leqslant 1{\cdot}0$$

c) *Stocky struts* — A strut not covered in (b) above shall satisfy

$$\frac{M_{pc}}{M_p} \leqslant 1$$

where

P = an axial force, compressive or tensile in a member;

M_{pc} = maximum moment (plastic) capacity acting in the beam-column;

M_p = plastic moment capacity of the section;

M_0 = lateral buckling strength in the absence of axial load
= M_p if the beam column is laterally braced;

P_{ac} = buckling strength in the plane of bending if axially loaded (without any bending moment) and if the beam column is laterally braced, as per **9.2.5.1**;

P_e = Euler load = $\frac{\pi^2 E A_s}{(L/r)^2}$ for the plane of bending;

P_y = yield strength of axially loaded section = $A_s \cdot f_y$;

A_s = effective cross-section area of the member;

C_m = a coefficient as defined in **7.1.3**;

r = radius of gyration about the same axis as the applied moment;

λ_0 = characteristic slenderness ratio

$$= \sqrt{\frac{P_y}{P_e}} = \frac{L}{\pi r}\sqrt{\frac{f_y}{E}};$$

β = ratio of end moment, each measured in the same rotational direction and chosen with the numerically large amount in the denominator (β range from + 1 for double curvature, 0 for one end pinned, to − 1 for single curvature); and

L = actual strut length.

9.2.6.2 A member assumed to contain plastic hinges and subjected to combined bending and axial compression with P/P_y not exceeding 0·15 shall have a value of P/P_y not exceeding $\frac{0.6 + 0.4\,\beta}{\lambda_0}$ where λ_0 and β are as defined above.

9.2.7 *Shear* — The calculated maximum shear capacity V_y of a beam or a beam-column shall be

$$V_y = 0.55\, A_w f_y$$

where A_w is the effective cross-sectional area resisting shear for calculating the average shear stress or the maximum shear capacity of the members.

9.2.8 *Stability* — The elastic buckling load of a frame or its components designed on the basis of plastic theory shall be at least three times the plastic collapse load. If an accurate estimate of the elastic buckling load is not available, this provision shall be deemed to be satisfied for frames of up to three storeys if the compressive force P, in each member does not exceed:

$$0{\cdot}33\ \frac{\pi^2 EI}{l^2}$$

for buckling in any direction, where the effective length l is determined according to **5.2**.

For frames of over three storeys, the calculated plastic collapse load shall include an assessment of the moment caused by the possible combination of high axial force and transverse deflection.

9.2.9 *Minimum Thickness*

9.2.9.1 *Compression Outstands* — A flange or other compression element required to participate in a plastic hinge shall not project beyond its outer most point of attachment by more than $136.T_1/\sqrt{f_y}$.

Where T_1 is the thickness of flange of a section or plate in compression or the aggregate thickness of plates if connected in accordance with Section 8.

For the purpose of this clause, web stiffeners at plastic hinges shall be proportioned as compression elements.

9.2.9.2 *Unsupported widths* — The distance between adjacent parallel lines of attachment of a compression flange or another compression element to other parts of member, when such flanges or elements are required to participate in a plastic hinge action, shall not exceed $512.\ T_1/\sqrt{f_y}$. Where T_1 is as defined in **9.2.9.1**.

9.2.9.3 *Webs in shear* — If the depth d_1 of a web subjected to shear and required to participate in a plastic hinge exceeds $\dfrac{688.t}{\sqrt{f_y}}$ then the compressive axial force P on the member shall not exceed the value

$$P = P_y\left(0{\cdot}70 - \frac{d_1}{t} \times \frac{\sqrt{f_y}}{1\,600}\right)$$

The maximum permissible value of d_1 in any plastic hinge zone shall be $\dfrac{1120.t}{\sqrt{f_y}}$.

9.2.9.4 *Web under bending and compression* — When the web is subjected to bending and compression, the following conditions shall be satisfied:

a) Where $\dfrac{P}{P_y}$ exceeds $0{\cdot}27$, then the depth d_1 shall not exceed $\dfrac{688.t}{\sqrt{f_y}}$; and

b) When $\frac{P}{P_y}$ is less than or equal to 0·27, then the depth d_1 shall not exceed $\left[\frac{1120}{\sqrt{f_y}} - \frac{1600}{\sqrt{f_y}}\left(\frac{P}{P_y}\right)\right] t$.

9.2.10 *Lateral Bracing*

9.2.10.1 Members shall be adequately braced to resist lateral and torsional displacement at the plastic hinge locations associated with failure mechanism. Lateral bracing mass be dispensed within the region of the last hinge to form in the failure mechanism assumed as the basis for proportioning the given member.

9.2.10.2 a) If the length along the member in which the applied moment exceeds 0·85. M_p, is less than or equal to $\frac{640 \nu . r_y}{\sqrt{f_y}}$, at least one critical flange support shall be provided within or at the end of this length and the spacing of the adjacent supports shall not exceed $960 \nu . r_y / \sqrt{f_y}$.

b) If the length along the membei in which the applied moment exceeds 0·85 M_p is greater than or equal to $\frac{640 \nu . r_y}{\sqrt{f_y}}$, the critical flange shall be supported in such a manner that no portion of this length is unsupported for a distance of more than $\frac{640 . \nu . r_y}{\sqrt{f_y}}$.

c) Lateral restraints for the remaining elastic portions of the member shall be designed in accordance with Sections 4 and 5 as appropiate, using stresses derived from the plastic bending moments multiplied by 1·7.

In this clause M_p shall be assumed as M_p or M_{pc} as appropriate.

ν may be taken as unity or calculated by the following expression:

$$\nu = \frac{1 \cdot 5}{\sqrt{1 + (\theta / 8)}}$$

where θ is the ratio of the rotation at the hinge point to the relative elastic rotation of the far ends of the beam segment containing the plastic hinge.

NOTE — The lateral restraints provided by this clause will ensure that a section delivers its full moment and deformation capacity. This may be too great for some design circumstances. With the approval of the appropriate authority the design engineer may use the methods which allow a reduced amount of bracing to be used, provided that this reduction is justified by rational and widely accepted means and that any associated reductions in moment and deformation capacity are fully considered in the design.

9.2.11 *Web Stiffening*

9.2.11.1 *Excessive shear forces* — Web stiffeners or doubler plates shall be provided when the requirements of **9.2.7** are not met, in which case the stiffeners or doubler plates shall be capable of carrying that portion of the force which exceeds the shear capacity of the web.

9.2.11.2 *Concentrated loads* — Web stiffeners shall be provided at points on a member where the concentrated force delivered by the flanges of another member framing into it will produce web crippling opposite the compression flange or high tensile stress in the connection of the tension flange. This requirement shall be deemed to be satisfied if web stiffeners are placed:

a) opposite the compression flange of the other member when

$$t < \frac{A_f}{T_b + 5k}$$

b) opposite the tension flange of the other member when

$$T_f < 0.4 . \sqrt{A_f}$$

where

t = thickness of web to be stiffened,

k = distance from outer face of flange to web toe of fillet of member to be stiffened,

T_b = thickness of flange delivering concentrated load,

T_f = thickness of flange of member to be stiffened, and

A_f = area of flange delivering concentrated load.

The area of such stiffeners, A_{st}, shall be such that

$$A_{st} \geqslant A_f - t\,(\,T_b + 5k\,)$$

The ends of such stiffeners shall be fully butt welded to the inside face of the flange adjacent to the concentrated tensile force. It shall be permissible to fit the stiffeners against the inside face of the flange adjacent to the concentrated compression force without welding. When the concentrated force is delivered by only one beam connected to an outside face of a strut, the length of the web stiffener shall extend for at least half the depth of the member, and the welding connecting it to the web shall be sufficient to develop a force of $f_y . A_{st}$.

9.2.11.3 *Plastic hinges* — Web stiffeners shall be provided at all plastic hinges where the applied load exceeds 0·06 $A_w f_y$, where A_w is as explained in **9.2.7**.

9.2.12 *Load Capacities of Connections* — The calculated load capacities of welds, bolts and rivets shall be taken as 1·7 times the values calculated using permissible stress specified in **8.9.4**.

9.3 Connections and Fabrication

9.3.1 *Connections*

9.3.1.1 All connections which are essential to the continuity, assumed as the basis of the design analysis shall be capable of resisting the moments, shears and axial loads to which they would be subjected by either full or factored loading.

9.3.1.2 Corner connections (haunches), tappered or curved for architectural reasons shall be so proportioned that the full plastic bending strength of the section adjacent to the connection may be developed.

9.3.1.3 Stiffeners shall be used, as required, to preserve the flange continuity of interrupted members at their junction with other members in a continuous frame. Such stiffeners shall be placed in pairs on opposite sides of the web of the member which extends continuously through the joint.

9.3.2 *Fabrication* — The provisions of Section 11 with respect to workmanship shall govern the fabrication of structures, or portions of structures, designed on the basis of maximum strength, subject to the following limitations:

a) The use of sheared edges shall be avoided in locations subject to plastic hinge rotation at factored loading. If used they shall be finished smooth by grinding, chipping or planing.

b) In locations subject to plastic hinge rotation at factored loading, holes for rivets or bolts in the tension area shall be sub-punched and reamed or drilled full size.

SECTION 10 DESIGN OF ENCASED MEMBERS

10.1 Encased Columns

10.1.1 *Conditions of Design* — A member may be designed as an encased column when the following conditions are fulfilled:

a) The member is of symmetrical I-shape or a single I-beam for channels back-to-back, with or without flange plates;

b) The overall dimensions of the steel section do not exceed 750 × 450 mm over plating where used, the larger dimension being measured parallel to the web;

c) The column is unpainted and is solidly encased in ordinary dense concrete, with 20 mm aggregate (unless solidity can be obtained with a larger aggregate) and of grade designation M 15, *Min* (*see* IS : 456-1978).

d) The minimum width of solid casing is equal to $b_0 + 100$ mm, where b_0 is the width of the steel flange in millimetres;

e) The surface and edges of the steel column have a concrete cover of not less than 50 mm;

f) The casing is effectively reinforced with steel wires. The wire shall be at least 5 mm in diameter and the reinforcement shall be in the form of stirrups or binding at not more than 150 mm pitch so arranged as to pass through the centre of the covering of the edges and outer faces of the flanges and supported by longitudinal spacing bars not less than four in number; and

g) Steel cores in encased columns shall be accurately machined at splices and provisions shall be made for alignment of column. At the column base provision shall be made to transfer the load to the footing at safe unit stresses in accordance with IS : 456-1978.

10.1.2 *Design of Member*

10.1.2.1 The steel section shall be considered as carrying the entire load but allowance may be made by assuming the radius of gyration '*r*' of the column section about the axis in the plane of its web to be $0{\cdot}2\ (b_0 + 100)$ mm, where b_0 is the width of the steel flange in millimetres. The radius of gyration about its other axis shall be taken as that of the uncased section.

10.1.2.2 The axial load on the encased column shall not exceed 2 times that which would be permitted on the uncased section, nor shall the slenderness ratio of the uncased section for its full length centre-to-centre of connections exceed 250.

10.1.2.3 In computing the allowable axial load on the encased strut, the concrete shall be taken as assisting in carrying the load over its rectangular cross section, any cover in excess of 75 mm from the overall dimensions of the steel section of the cased strut being ignored.

10.1.2.4 The allowable compressive load P in case of encased columns shall be determined as follows:

$$P = A_{sc}\sigma_{sc} + A_c\sigma_c$$

where

A_{sc}, A_c = cross-sectional area of steel and concrete, and

σ_{sc}, σ_c = permissible stresses in steel and concrete in compression.

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

10.2 Encased Beams

10.2.1 *Conditions of Design* — Beams and girders with equal flanges may be designed as encased beams when the following conditions are fulfilled:

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

b) The beam is unpainted and is solidly encased in ordinary dense concrete, with 10 mm aggregate (unless solidity can be obtained with a larger aggregate), and of a grade designation M 15, *Min* (*see* IS : 456-1978);

c) The minimum width of solid casing = (b_0 + 100) mm, where b_0 is the width of the steel flange in mm;

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

e) The casing is effectively reinforced with steel wire of at least 5 mm diameter and the reinforcement shall be in the form of stirrups or binding at not more than 150 mm pitch, and so arranged as to pass through the centre of the covering to the edges and soffit of the lower flange.

10.2.2 *Design of Member* — The steel section shall be considered as carrying the entire load but allowance may be made for the effect of the concrete on the lateral stability of the compression flange. This allowance should be made by assuming for the purpose of determining the permissible stress in compression that the equivalent moment of inertia (I_y) about the *y-y* axis is equal to $A.r_y^2$, where A is the area of steel section and r_y may be taken as 0·2 (b_0 + 100) mm. Other properties required for referring to **6.2** may be taken as for the uncased section. The permissible bending stress so determined shall not exceed 1·5 times that permitted for the uncased section.

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

SECTION 11 FABRICATION AND ERECTION

11.1 General — Tolerances for fabrication of steel structures shall conform to IS : 7215-1974. Tolerances for erection of steel structures shall conform to the Indian Standard.* For general guidance on fabrication by welding, reference may be made to IS : 9595-1980.

11.2 Fabrication Procedures

11.2.1 *Straightening* — All material shall be straight and, if necessary, before being worked shall be straightened and/or flattened by pressure, unless required to be of curvilinear form and shall be free from twists.

11.2.2 *Clearances* — The erection clearance for cleated ends of members connecting steel to steel should preferably be not greater than 2·0 mm at each end. The erection clearance at ends of beams without web cleats should be not more than 3 mm at each end, but where, for practical reasons, greater clearance is necessary, suitably designed seatings should be provided.

11.2.2.1 Where black bolts are used, the holes may be made not more than 1·5 mm greater than the diameter of the bolts, unless otherwise specified by the engineer.

11.2.3 *Cutting*

11.2.3.1 Cutting may be effected by shearing, cropping or sawing. Gas cutting by mechanically controlled torch may be permitted for mild steel only. Gas cutting of high tensile steel may also be permitted provided special care is taken to leave sufficient metal to be removed by machining so that all metal that has been hardened by flame is removed. Hand flame cutting may be permitted subject to the approval of the inspector.

11.2.3.2 Except where the material is subsequently joined by welding, no loads shall be transmitted into metal through a gas cut surface.

11.2.3.3 Shearing, cropping and gas cutting, shall be clean, reasonably square, and free from any distortion, and should the inspector find it necessary, the edges shall be ground afterwards.

11.2.4 *Holing*

11.2.4.1 Holes through more than one thickness of material for members, such as compound stanchion and girder flanges shall, where possible, be drilled after the members are assembled and tightly clamped or bolted together. Punching may be permitted before assembly, provided

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*Tolerances for erection of steel structures (*under preparation*).

the holes are punched 3 mm less in diameter than the required size and reamed after assembly to the full diameter. The thickness of material punched shall be not greater than 16 mm. For dynamically loaded structures, punching shall be avoided.

11.2.4.2 When holes are drilled in one operation through two or more separable parts, these parts, when so specified by the engineer, shall be separated after drilling and the burrs removed.

11.2.4.3 Holes in connecting angles and plates, other than splices, also in roof members and light framing, may be punched full size through material not over 12 mm thick, except where required for close tolerance bolts or barrel bolts.

11.2.4.4 Matching holes for rivets and black bolts shall register with each other so that a gauge of 1·5 mm or 2·0 mm (as the case may be depending on whether the diameter of the rivet or bolt is less than or more than 25 mm) less in diameter than the diameter of the hole will pass freely through the assembled members in the direction at right angle to such members. Finished holes shall be not more than 1·5 mm or 2·0 mm (as the case may be) in diameter larger than the diameter of the rivet or black bolt passing through them, unless otherwise specified by the engineer.

11.2.4.5 Holes for turned and fitted bolts shall be drilled to a diameter equal to the nominal diameter of the shank or barrel subject to H8 tolerance specified in IS : 919-1963. Preferably parts to be connected with close tolerance or barrel bolts shall be firmly held together by tacking bolts or clamps and the holes drilled through all the thicknesses at one operation and subsequently reamed to size. All holes not drilled through all thicknesses at one operation shall be drilled to a smaller size and reamed out after assembly. Where this is not practicable, the parts shall be drilled and reamed separately through hard bushed steel jigs.

11.2.4.6 Holes for rivets or bolts shall not be formed by gas cutting process.

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

11.4 Riveting

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

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

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

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

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

11.5 Bolting

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

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

11.5.3 In all cases where the full bearing area of the bolt is to be developed, the bolt shall be provided with a washer of sufficient thickness under the nut to avoid any threaded portion of the bolt being within the thickness or the parts bolted together.

11.6 Welding

11.6.1 Welding shall be in accordance with IS : 816-1969, IS : 819-1957, IS : 1024-1979, IS : 1261-1959, IS : 1323-1982 and IS : 9595-1980, as appropriate.

11.6.2 For welding of any particular type of joint, welders shall give evidence acceptable to the purchaser of having satisfactorily completed appropriate tests as described in any of the Indian Standards — IS : 817-1966, IS : 1393-1961, IS : 7307 (Part 1)-1974, IS : 7310 (Part 1)-1974 and IS : 7318 (Part 1)-1974, as relevant.

11.7 Machining of Butts, Caps and Bases

11.7.1 Column splices and butt joints of struts and compression members depending on contact for stress transmission shall be accurately machined and close-butted over the whole section with a clearance not exceeding 0·2 mm locally at any place. In column caps and bases, the ends of shafts together with the attached gussets, angles, channels, etc, after riveting together should be accurately machined so that the parts connected butt over the entire surfaces of contact. Care should be taken that these gussets, connecting angles or channels are fixed with such accuracy that they are not reduced in thickness by machining by more than 2·0 mm.

11.7.2 Where sufficient gussets and rivets or welds are provided to transmit the entire loading (*see* Section 5) the column ends need not be machined.

11.7.3 Ends of all bearing stiffeners shall be machined or ground to fit tightly at both top and bottom.

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

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

11.8 Solid Round Steel Columns

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

11.8.2 The tolerance between the reduced end of the shaft and the hole, in the case of slabs welded in position, shall not exceed 0·25 mm.

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

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

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

11.9 Painting

11.9.1 Painting shall be done in accordance with IS : 1477 (Part 1)-1971 and IS : 1477 (Part 2)-1971.

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

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

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

11.9.5 Chequered plates shall be painted but the details of painting shall be specified by the purchaser.

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

11.9.7 Welds and adjacent parent metal shall not be painted prior to deslagging, inspection and approval.

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

11.10 Marking

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

11.11 Shop Erection

11.11.1 The steelwork shall be temporarily shop erected complete or as arranged with the inspector so that accuracy of fit may be checked before despatch. The parts shall be shop assembled with sufficient numbers of parallel drifts to bring and keep the parts in place.

11.11.2 In the case of parts drilled or punched, through steel jigs with bushes resulting in all similar parts being interchangeable, the steelwork may be shop erected in such position as arranged with the inspector.

11.12 Packing — All projecting plates or bars and all ends of members at joints shall be stiffened, all straight bars and plates shall be bundled, all screwed ends and machined surfaces shall be suitably packed and all rivets, bolts, nuts, washers and small loose parts shall be packed separately in cases so as to prevent damage or distortion during transit.

11.13 Inspection and Testing

11.13.1 The inspector shall have free access at all reasonable times to those parts of the manufacturer's works which are concerned with the fabrication of the steelwork and shall be afforded all reasonable facilities for satisfying himself that the fabrication is being undertaken in accordance with the provisions of this standard.

11.13.2 Unless specified otherwise, inspection shall be made at the place of manufacture prior to despatch and shall be conducted so as not to interfere unnecessary with the operation of the work.

11.13.3 The manufacturer shall guarantee compliance with the provisions of this standard, if required to do so by the purchaser.

11.13.4 Should any structure or part of a structure be found not to comply with any of the provisions of this standard, it shall be liable to rejection. No structure or part of the structure, once rejected shall be resubmitted for test, except in cases where the purchaser or his authorised representative considers the defect as rectifiable.

11.13.5 Defects which may appear during fabrication shall be made good with the consent of and according to the procedure laid down by the inspector.

11.13.6 All gauges and templates necessary to satisfy the inspector shall be supplied by the manufacturer. The inspector, may, at his discretion, check the test results obtained at the manufacturer's works by independent tests at the Government Test House or elsewhere, and should the material so tested be found to be unsatisfactory, the costs of such tests shall be borne by the manufacturer, and if satisfactory, the costs shall be borne by the purchaser.

11.14 Site Erection

11.14.1 *Plant and Equipment* — The suitability and capacity of all plant and equipment used for erection shall be to the satisfaction of the engineer.

11.14.2 *Storing and Handling* — All structural steel should be so stored and handled at the site that the members are not subjected to excessive stresses and damage.

11.14.3 *Setting Out* — The positioning and levelling of all steelwork, the plumbing of stanchions and the placing of every part of the structure with accuracy shall be in accordance with the approved drawings and to the satisfaction of the engineer.

11.14.4 *Security During Erection*

11.14.4.1 For safety precautions during erection of steel structures reference shall be made to IS : 7205-1973.

11.14.4.2 During erection, the steelwork shall be securely bolted or otherwise fastened and, when necessary, temporarily braced to provide for all load to be carried by the structure during erection including those due to erection equipment and its operation.

11.14.4.3 No riveting, permanent bolting or welding should be done until proper alignment has been obtained.

11.14.5 *Field Connections*

11.14.5.1 *Field riveting* — Rivets driven at the site shall be heated and driven with the same care as those driven in the shop.

11.14.5.2 *Field bolting* — Field bolting shall be carried out with the same care as required for shop bolting.

11.14.5.3 *Field welding* — All field assembly and welding shall be executed in accordance with the requirements for shop fabrication excepting such as manifestly apply to shop conditions only. Where the steel has been delivered painted, the paint shall be removed before field welding, for a distance of at least 50 mm on either side of the joint.

11.15 Painting After Erection

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

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

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

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

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

11.16 Bedding of Stanchion Bases and Bearings of Beams and Girders on Stone, Brick or Concrete (Plain or Reinforced)

11.16.1 Bedding shall be carried out with portland cement, grout or mortar, as described under **11.16.4** or fine cement concrete in accordance with IS : 456-1978.

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

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

11.16.4 Bedding of structure shall be carried out with grout or mortar which shall be of adequate strength and shall completely fill the space to be grouted and shall either be placed under pressure or by ramming against fixed supports.

SECTION 12 STEEL-WORK TENDERS AND CONTRACTS

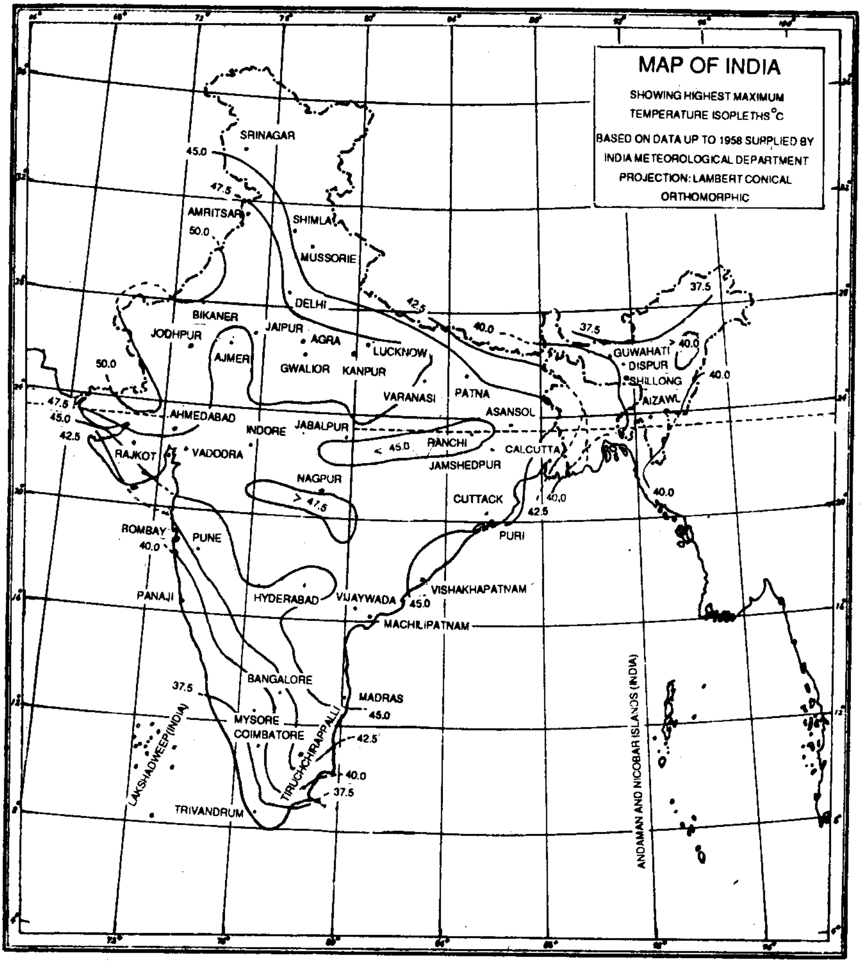
12.1 General Recommendations

12.1.1 A few recommendations are given in Appendix G for general information.

APPENDIX A

(*Clause* 3.3.2)

CHART SHOWING HIGHEST MAXIMUM TEMPERATURE



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

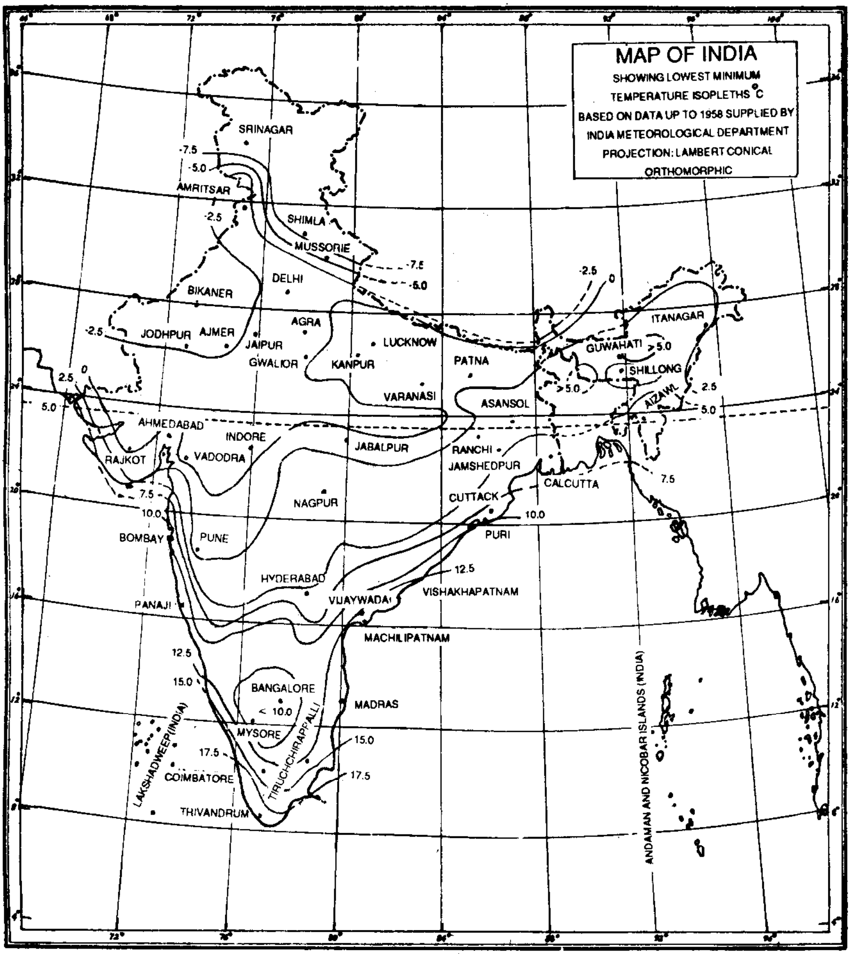
Based upon Survey of India map with the permission of the Surveyor General of India.

Responsibility for the correctness of internal details rests with the publishers.

APPENDIX B

(*Clause* 3.3.2)

CHART SHOWING LOWEST MINIMUM TEMPERATURE



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

Based upon Survey of India map with the permission of the Surveyor General of India.

Responsibility for the correctness of internal details rests with the publishers.

APPENDIX C

(*Clause* 5.2.2)

EFFECTIVE LENGTH OF COLUMNS

C-1. In the absence of more exact analysis, the effective length of columns in framed structures may be obtained from the ratio l/L, of effective length l to unsupported length L given in Fig. C-1 when relative displacement of the ends of the column is prevented and in Fig. C-2 when relative lateral displacement of the ends is not prevented. In the later case, it is recommended that the effective length ratio l/L may not be taken to be less than 1.2.

In Fig. C-1 and Fig. C-2, β_1 and β_2 are equal to $\dfrac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$

where the summation is to be done for the members framing into a joint at top and bottom respectively; K_c and K_b being the flexural stiffnesses for the column and beam, respectively.

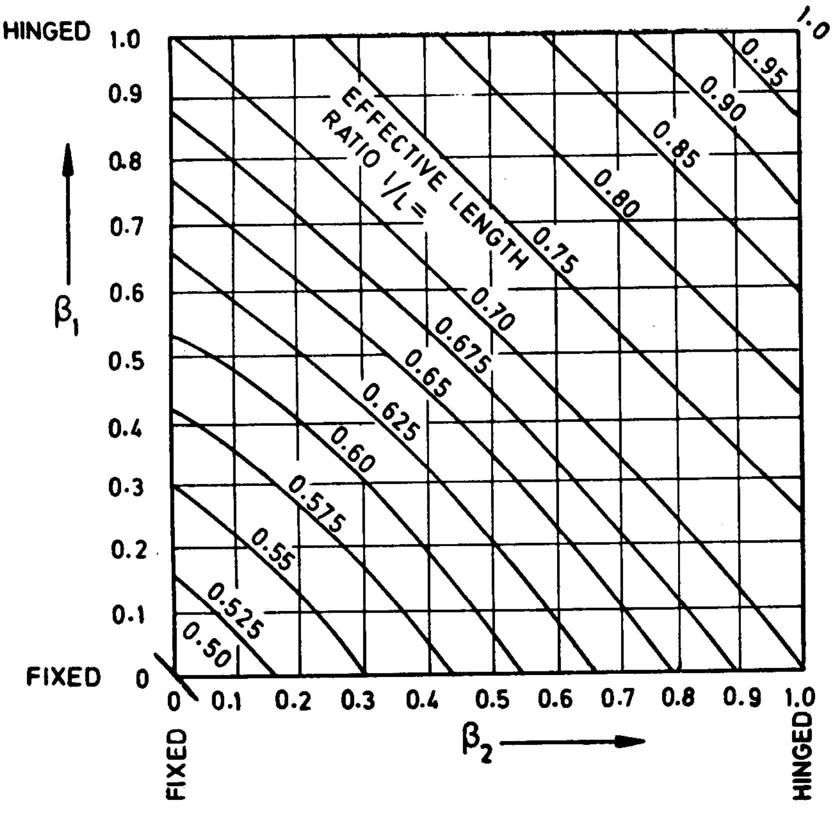


FIG. C-1 EFFECTIVE LENGTH RATIOS FOR A COLUMN IN A FRAME WITH NO SWAY

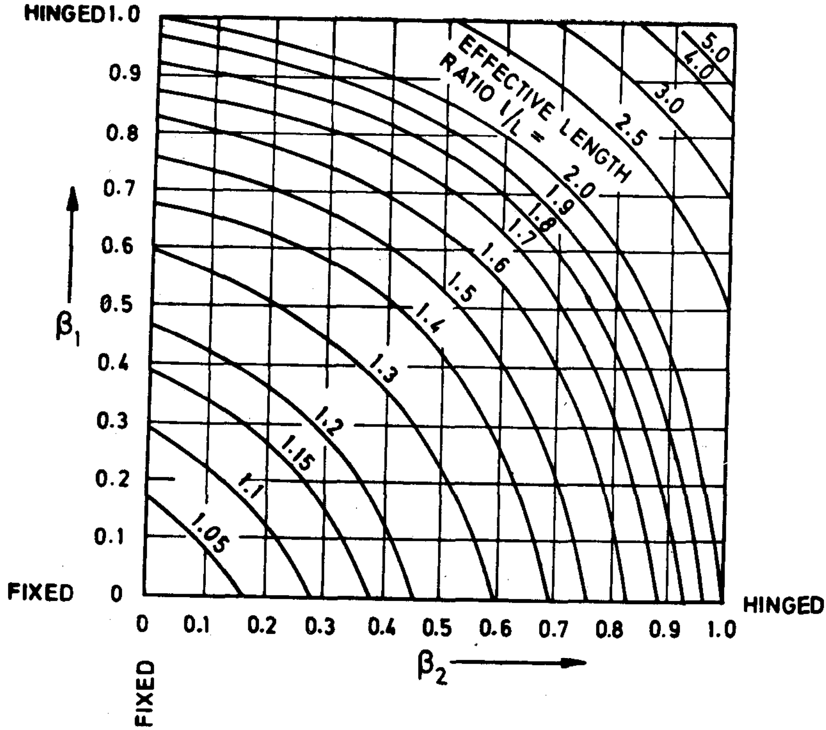


FIG. C-2 EFFECTIVE LENGTH RATIOS FOR A COLUMN IN A FRAME WITHOUT RESTRAINT AGAINST SWAY

APPENDIX D

(*Clause* 5.2.5)

METHOD FOR DETERMINING EFFECTIVE LENGTH FOR STEPPED COLUMNS

D-1. SINGLE STEPPED COLUMNS

D-1.1 Effective lengths in the plane of stepping (bending about axis *x-x*) for bottom and top parts for single stepped columns shall be taken as given in Table D-1.

NOTE — The provisions of **D-1.1** are applicable to intermediate columns as well with steppings on either side, provided appropriate values of I_1 and I_2 are taken.

TABLE D-1 EFFECTIVE LENGTH OF SINGLED STEPPED COLUMNS

(*Clause* D-1.1)

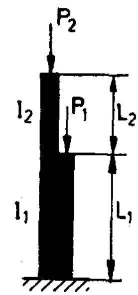
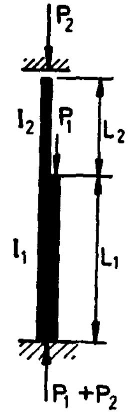
Sl No. (1)	Degree of End Restraint (2)	Sketch (3)	Effective Length Coefficients (4)	Column Parameters for All Cases (5)
a)	Effectively held in position and restrained against rotation at both ends		$k_1=\sqrt{\frac{k_{12}^2+k_{11}^2(\alpha-1)}{\alpha}}$ $k_2=\frac{k_1}{C_1} < 3$ where k_{12} and k_{11} are to be taken as per Table D-2	
b)	Effectively held in position at both ends and restrained against rotation at bottom end only		$k_1=\sqrt{\frac{k_{12}^2+k_{11}^2(\alpha-1)}{\alpha}}$ $k_2=\frac{k_1}{C_1} < 3$ where k_{12} and k_{11} are to be taken as per Table D-3	$\alpha=\frac{P_1+P_2}{P_2}$ $C_1=\frac{L_2}{L_1}\sqrt{\frac{I_1}{I_2\alpha}}$ $\frac{i_2}{i_1}=\frac{I_2}{L_2}\times\frac{L_1}{I_1}$
c)	Effectively held in position and restrained against rotation at bottom end, and top end held against rotation but not held in position		k_1 to be taken as per Table D-4 $k_2=\frac{k_1}{C_1} < 3$	Effective length of bottom part of column in plane of stepping $= k_1 L_1$ Effective length of top part of column in plane of stepping $= k_2 L_2$
d)	Effectively held in position and restrained against rotation at bottom end, and top end neither held against rotation nor held in position		k_1 to be taken as per Table D-5 $k_2=\frac{k_1}{C_1} < 3$	

TABLE D-2 COEFFICIENTS OF EFFECTIVE LENGTHS $\bar{k}_{12}$ AND $\bar{k}_{11}$ FOR COLUMNS WITH BOTH ENDS EFFECTIVELY HELD IN POSITION AND RESTRAINED AGAINST ROTATION

(*Table* D-1)

$\frac{I_2}{I_1}$	Coefficients $\bar{k}_{12}$ and $\bar{k}_{11}$ for L_2/L_1 Equal to														
	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0	1·2	1·4	1·5	1·8	2·0
	Coefficient $\bar{k}_{12}$ ($P_1 = 0$)														
0·05	0·74	0·94	1·38	1·60	1·87	2·07	2·23	2·39	2·52	2·67	3·03	3·44	3·85	4·34	4·77
0·1	0·67	0·76	1·00	1·20	1·42	1·61	1·78	1·92	2·04	2·20	2·40	2·60	2·86	3·18	3·41
0·2	0·64	0·70	0·79	0·93	1·07	1·23	1·41	1·50	1·60	1·72	1·92	2·11	2·28	2·45	2·64
0·3	0·62	0·68	0·74	0·85	0·95	1·06	1·18	1·28	1·39	1·48	1·67	1·82	1·96	2·12	2·20
0·4	0·60	0·66	0·71	0·78	0·87	0·99	1·07	1·16	1·26	1·34	1·50	1·65	1·79	1·94	2·08
0.5	0·59	0·65	0·70	0·77	0·82	0·93	0·99	1·08	1·17	1·23	1·39	1·53	1·66	1·79	1·92
1·0	0·55	0·60	0·65	0·70	0·75	0·80	0·85	0·90	0·95	1·00	1·10	1·20	1·30	1·40	1·50
	Coefficient $\bar{k}_{11}$ ($P_2 = 0$)														
0·05	0·65	0·67	0·71	0·85	1·01	1·17	1·31	1·41	1·50	1·57	1·67	1·74	1·78	1·82	1·86
0·1	0·64	0·65	0·65	0·65	0·78	0·92	1·05	1·15	1·25	1·33	1·45	1·55	1·62	1·68	1·71
0·2	0·62	0·64	0·65	0·65	0·66	0·73	0·83	0·92	1·01	1·09	1·23	1·33	1·41	1·48	1·54
0·3	0·60	0·63	0·64	0·65	0·66	0·67	0·73	0·81	0·89	0·94	1·09	1·20	1·28	1·35	1·41
0·4	0·58	0·63	0·63	0·64	0·64	0·66	0·68	0·75	0·82	0·88	1·01	1·10	1·19	1·26	1·32
0·5	0·57	0·61	0·63	0·64	0·64	0·65	0·68	0·72	0·77	0·83	0·94	1·04	1·12	1·19	1·25
1·0	0·55	0·58	0·60	0·61	0·62	0·63	0·65	0·67	0·70	0·73	0·80	0·88	0·93	1·01	1·05

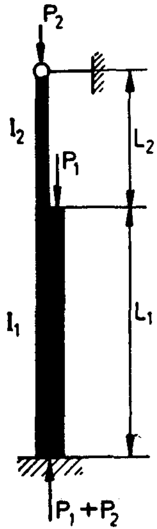


Note — Intermediate value may be obtained by interpolation.

TABLE D-3 COEFFICIENTS OF EFFECTIVE LENGTHS $\bar{k}_{12}$ AND $\bar{k}_{11}$ FOR COLUMNS WITH BOTH ENDS EFFECTIVELY HELD IN POSITION AND RESTRAINED AGAINST ROTATION AT BOTTOM END ONLY

(*Table* D-1)

I_2/I_1	Coefficients $\bar{k}_{12}$ and $\bar{k}_{11}$ for L_2/L_1 Equal to														
	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0	1·2	1·4	1·6	1·8	2·0
	Coefficient $\bar{k}_{12}$ ($P_1 = 0$)														
0·05	0·97	1·66	2·10	2·43	2·72	2·92	3·08	3·24	3·48	3·73	4·22	4·85	5·36	6·00	6·58
0·1	0·83	1·21	1·57	1·95	2·14	2·33	2·46	2·60	2·76	2·91	3·28	3·61	4·03	4·43	4·85
0·3	0·78	0·90	1·09	1·27	1·44	1·60	1·74	1·86	1·98	2·11	2·35	2·51	2·76	2·99	3·25
0·5	0·78	0·86	0·99	1·10	1·22	1·35	1·47	1·57	1·67	1·76	1·96	2·15	2·34	2·50	2·76
1·0	0·78	0·85	0·92	0·99	1·06	1·13	1·20	1·27	1·34	1·41	1·54	1·68	1·82	1·97	2·10
	Coefficient $\bar{k}_{11}$ ($P_2 = 0$)														
0·05	0·67	0·67	0·82	1·16	1·35	1·48	1·58	1·65	1·69	1·74	1·81	1·84	1·86	1·88	1·90
0·1	0·67	0.67	0·73	0·93	1·11	1·25	1·36	1·45	1·52	1·57	1·66	1·72	1·77	1·80	1·82
0·3	0·67	0·67	0·67	0·71	0·80	0·90	0·99	1·08	1·15	1·22	1·33	1·41	1·48	1·54	1·59
0·5	0·67	0·67	0·67	0·69	0·73	0·81	0·87	0·94	1·01	1·07	1·17	1·26	1·33	1·39	1·44
1·0	0·67	0·67	0·67	0·68	0·71	0·74	0·78	0·82	0·87	0·91	0·99	1·07	1·13	1·19	1·24

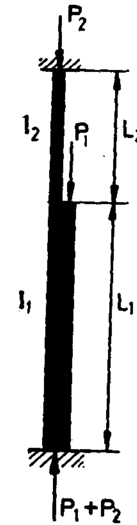


NOTE — Intermediate values may be obtained by interpolation.

TABLE D-4 COEFFICIENT OF EFFECTIVE LENGTH $\bar{k}_1$ FOR COLUMNS EFFECTIVELY HELD IN POSITION AND RESTRAINED AGAINST ROTATION AT BOTTOM END AND TOP END HELD AGAINST ROTATION BUT NOT HELD IN POSITION

(*Table* D-1)

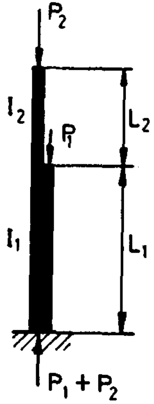
C_1	Coefficient $\bar{k}_1$ for i_2/i_1 Equal to														
	0	0·2	0·4	0·6	0·8	1·0	1·2	1·4	1·6	1·8	2·0	2·5	5·0	10	20
0	2·0	1·86	1·76	1·67	1·60	1·55	1·50	1·46	1·43	1·40	1·37	1·32	1·18	1·10	1·05
0·5	2·0	1·90	1·80	1·74	1·69	1·65	1·61	1·58	1·55	1·53	—	—	—	—	—
1·0	2·0	2·00	2·00	2·00	2·00	2·00	—	—	—	—	—	—	—	—	—
1·5	2·0	2·25	2·38	2·48	—	—	—	—	—	—	—	—	—	—	—
2·0	2·0	2·66	2·91	—	—	—	—	—	—	—	—	—	—	—	—
2·5	2·5	3·17	3·50	—	—	—	—	—	—	—	—	—	—	—	—
3·0	3·0	3·70	4·12	—	—	—	—	—	—	—	—	—	—	—	—



NOTE — Intermediate values may be obtained by interpolation.

TABLE D-5 COEFFICIENT OF EFFECTIVE LENGTH $\bar{k}_1$ FOR COLUMNS WITH TOP END FREE AND BOTTOM END EFFECTIVELY HELD IN POSITION AND RESTRAINED AGAINST ROTATION

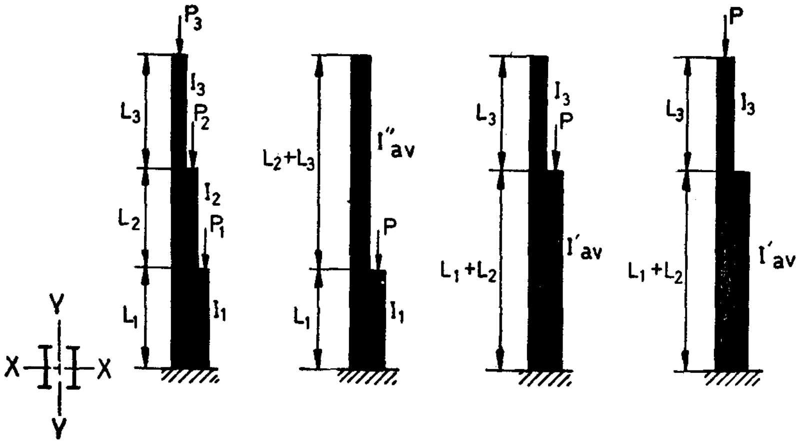
C_1	Coefficient $\bar{k}_1$ for i_2/i_1 Equal to														
	0	0·2	0·4	0·6	0·8	1·0	1·2	1·4	1·6	1·8	2·0	2·5	5·0	10	20
0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0	2·0
0·5	2·0	2·14	2·24	2·36	2·47	2·57	2·67	2·76	2·85	2·94	3·02	—	—	—	—
1·0	2·0	2·73	3·13	3·44	3·74	4·00	—	—	—	—	—	—	—	—	—
1·5	3·0	3·77	4·35	4·86	—	—	—	—	—	—	—	—	—	—	—
2·0	4·0	4·90	5·67	—	—	—	—	—	—	—	—	—	—	—	—
2·5	5·0	6·08	7·00	—	—	—	—	—	—	—	—	—	—	—	—
3·0	6·0	7·25	—	—	—	—	—	—	—	—	—	—	—	—	—



NOTE — Intermediate values may be obtained by interpolation.

D-2. EFFECTIVE LENGTH FOR DOUBLE STEPPED COLUMNS

D-2.1 Effective lengths in the plane of steppings (bending about axis *x-x*) for bottom, middle and top parts for a double stepped column shall be taken as follows:



Coefficient k_1 for effective length of bottom part of double stepped column shall be taken from the formula:

$$k_1 = \sqrt{\frac{t_1\bar{k}_1^2 + (t_2\bar{k}_2^2 + \bar{k}_3^2) \times (1 + n_2)^2 \times \frac{I_1}{I'_{av}}}{1 + t_1 + t_2}}$$

where

$\bar{k}_1, \bar{k}_2, \bar{k}_3$ are taken from Table D-6,

$t_1 = \frac{P_1}{P_3}$,

$t_2 = \frac{P_2}{P_3}$

$n_2 = \frac{L_2}{L_1}$,

I'_{av} = Average value of moment of inertia for the lower and middle parts

$= \frac{I_1L_1 + I_2L_2}{L_1 + L_2}$

I''_{av} = Average value of moment of inertia for middle and top parts

$$= \frac{I_2 L_2 + I_3 L_3}{L_2 + L_3}$$

Value of coefficient $\bar{k}_2$ for middle part of column is given by formula

$$k_2 = \frac{k_1}{C_2}$$

and coefficient k_3 for the top part of the column is given by

$$k_3 = \frac{k_1}{C_3} \leqslant 3$$

where

$$C_2 = \frac{L_2}{L_1}\sqrt{\frac{I_1 (P_2 + P_3)}{I_2 (P_1 + P_2 + P_3)}}$$

$$C_3 = \frac{L_3}{L_1}\sqrt{\frac{I_1 P_3}{I_3 (P_1 + P_2 + P_3)}}$$

NOTE — The provisions of **D-2.1** are applicable to intermediate columns as well with steppings on either side, provided appropriate values of I_1, I_2 and I_3 are taken.

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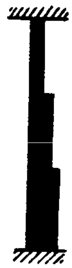
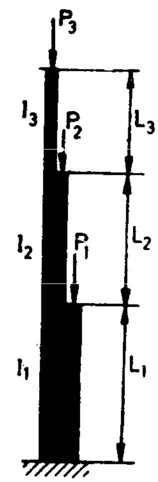
TABLE D-6 VALUES OF $\bar{k}_1$, $\bar{k}_2$ AND $\bar{k}_3$

(*Clause* D-2.1)

Sl No.	Degree of End Restraint	Sketch	$\bar{k}_1$	$\bar{k}_2$	$\bar{k}_3$	Column Parameters for All Cases
a)	Effectively held in position and restrained against rotation at both ends.		$\bar{k}_1 = \bar{k}_{11}$ where $\bar{k}_{11}$ is taken from Table D-2	$\bar{k}_2 = \bar{k}_{11}$ where $\bar{k}_{11}$ is taken from Table D-2	$\bar{k}_3 = \bar{k}_{12}$ where $\bar{k}_{12}$ is taken from Table D-2	
b)	Effectively held in position at both ends and restrained against rotation at bottom end only.		$\bar{k}_1 = \bar{k}_{11}$ where $\bar{k}_{11}$ is taken from Table D-3	$\bar{k}_2 = \bar{k}_{11}$ where $\bar{k}_{11}$ is taken from Table D-3	$\bar{k}_3 = \bar{k}_{12}$ where $\bar{k}_{12}$ is taken form Table D-3	
c)	Effectively held in position and restrained against rotation at bottom end, and top end held against rotation but not held in position.		$\bar{k}_1 = \bar{k}_1$ where $_1\bar{k}$ is taken from Table D-4 with $C_1 = 0$	$\bar{k}_2 = \bar{k}_1$ where $\bar{k}_1$ is taken from Table D-4 with $C_1 = 0$	$\bar{k}_3 = \bar{k}_1$ where $\bar{k}_1$ is taken from Table D-4 with $C_1 = \frac{L_3}{L_1 + L_3}\sqrt{\frac{I'_{av}}{I_3}}$	
d)	Effectively held in position and restrained against rotation at bottom end, and top end neither held against rotation nor against translation.		$\bar{k}_1 = 2$	$\bar{k}_2 = 2$	$\bar{k}_3 = \bar{k}_1$ where $\bar{k}_1$ is taken from Table D-5 with $C_1 = \frac{L_3}{L_1 + L_3}\sqrt{\frac{I'_{av}}{I_3}}$	

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APPENDIX E

(*Clause* 6.2.4.1)

LIST OF REFERENCES ON THE ELASTIC FLEXURAL TORSIONAL BUCKLING OF STEEL BEAMS

Textbooks

TIMOSHENKO (S P) and GERE (J M). Theory of elastic stability. Ed 2, 1961. McGraw-Hill, New York.

BLEICH (F), Buckling Strength of Metal Structures, 1952. McGraw-Hill, New York.

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GALAMBOS (T V). Structural members and frames. 1968. Prentice-Hall, New Jersey

Handbook of Structural Stability. Column Research Committee of Japan. 1971. Corona Publishing Co, Tokyo.

Allen (H G) and Bulson (P S). Background to buckling.

Stability of structure under static and dynamic loads. American Society of Civil Engineers, 1977 Ed.

References Prior to 1961

LEE (G C), A survey of literature on the lateral instability of beams. Welding Research Council Bulletin Series No. 63, Aug 1960.

Values of Elastic Critical Loads and Effective Length Factors

TRAHAIR (N S). The Bending stress rules of the draft AS CA 1. J Inst. Engrs Aust. 38; No. 6, June 1966.

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TRAHAIR (N S). Elastic stability of propped cantilevers. Civ Eng Trans, Institution of Engineers, Australia, V CE 10. No. 1; April 1968.

Safe load tables for laterally unsupported angles. Australian Institute of Steel Construction.

APPENDIX F

(*Clause* 9.2.3.2)

PLASTIC PROPERTIES OF INDIAN STANDARD MEDIUM WEIGHT BEAMS [IS : 808 (Part 1)-1973]

Designation	Weight	Sectional Area	Depth of Section (*D*)	Width of Flange (*b*)	Thickness of Flange (*T*)	Thickness of Web (*t*)	Radii of Gyration r_{xx}	Radii of Gyration r_{yy}	Section Modulus Z_x	Plastic Section Modulus Z_p	Shape Factor $\frac{Z_p}{Z_x}$	$\frac{b}{T}$	$\frac{D}{t}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	kg/m	cm^2	mm	mm	mm	mm	cm	cm	cm^3	cm^3			
ISMB 100	11·5	14·7	100	70	7·5	4·5	4·14	1·55	50·4	58·6	1·16	9·33	22·2
ISMB 125	13·4	17·0	125	70	8·0	5·0	5·16	1·51	71·2	81·8	1·15	8·75	25·0
ISMB 150	15	19·1	150	75	8·0	5·0	6·13	1·57	95·7	110	1·15	9·38	30·0
ISMB 175	19·5	24·9	175	85	9·0	5·8	7·13	1·76	144·3	166	1·15	9·44	30·17
ISMB 200	25·4	32·3	200	100	10·8	5·7	8·32	2·15	224	254	1·13	9·26	35·1
ISMB 225	31·2	39·7	225	110	11·8	6·5	9·31	2·34	306	348	1·14	9·32	34·6
ISMB 250	37·3	47·6	250	125	12·5	6·9	10·40	2·65	410	466	1·14	10·00	36·2
ISMB 300	46·1	58·7	300	140	13·1	7·7	12·40	2·86	599	683	1·14	10·7	38·9
ISMB 350	52·4	66·7	350	140	14·2	8·1	14·30	2·84	779	890	1·14	9·86	43·2
ISMB 400	61·6	78·5	400	140	16·0	8·9	16·2	2·82	1 020	1 176	1·15	8·75	44·9
ISMB 450	72·4	92·3	450	150	17·4	9·4	18·2	3·01	1 350	1 533	1·14	8·62	47·9
ISMB 500	86·9	111	500	180	17·2	10·2	20·2	3·52	1 810	2 075	1·15	10·47	49·0
ISMB 550	104	132	550	190	19·3	11·2	22·2	3·73	2 360	2 712	1·15	9·84	49·1
ISMB 600	123	156	600	210	20·8	12·0	24·2	4·12	3 060	3 511	1·15	10·10	50·0

APPENDIX G
(*Clause* 12.1.1)

GENERAL RECOMMENDATIONS FOR STEELWORK TENDERS AND CONTRACTS

G-0. GENERAL

G-0.1 The recommendations given in this Appendix are in line with those generally adopted for steelwork construction and are meant for general information.

G-0.2 These recommendations do not form part of the requirements of the standard and compliance with these is not necessary for the purpose of complying with this Code.

G-0.3 The recommendations are unsuitable for inclusion as a block requirement in a contract, but in drawing up a contract the points mentioned should be given consideration.

G-1. EXCHANGE OF INFORMATION

G-1.1 Before the steelwork design is commenced, the building designer should be satisfied that the planning of the building, its dimensions and other principal factors meet the requirements of the building owner and comply with regulations of all authorities concerned. Collaboration of building designer and steelwork designer should begin at the outset of the project by joint consideration of the planning and of such questions as the stanchion spacing, materials to be used for the construction, and depth of basement.

G-2. INFORMATION REQUIRED BY THE STEELWORK DESIGNER

G-2.1 General

a) Site plans showing in plan and elevation of the proposed location and main dimensions of the building or structure;

b) Ground levels, existing and proposed;

c) Particulars of buildings or other constructions which may have to remain on the actual site of the new building or structure during the erection of the steelwork;

d) Particulars of adjacent buildings affecting, or affected by, the new work;

e) Stipulation regarding the erection sequence or time schedule;

f) Conditions affecting the position or continuity of members;

g) Limits of length and weight of steel members in transit and erection;

h) Drawings of the substructure, proposed or existing, showing:

  i) levels of stanchion foundations, if already determined;

  ii) any details affecting the stanchion bases or anchor bolts;

  iii) permissible bearing pressure on the foundation; and

  iv) provisions for grouting (*see* **11.16**).

In the case of new work, the substructure should be designed in accordance with the relevant codes dealing with foundations and substructure;

j) The maximum wind velocity appropriate to the site (*see* IS : 875-1964); and

k) Environmental factors, such as proximity to sea coast, and corrosive atmosphere. Reference to bye-laws and regulations affecting the steelwork design and construction.

G-2.2 Further Information Relating to Buildings

a) Plans of the floors and roof with principal dimensions, elevations and cross sections showing heights between floor levels.

b) The occupancy of the floors and the positions of any special loads should be given.

c) The building drawings, which should be fully dimensioned, should preferably be to the scale of 1 to 100 and should show all stairs, fire-escapes, lifts, etc, suspended ceilings, flues and ducts for heating and ventilating. Doors and windows should be shown, as the openings may be taken into account in the computation of dead load.

Requirements should be given in respect of any maximum depth of beams or minimum head room.

Large-scale details should be given of any special features affecting the steelwork.

d) The inclusive weight per m^2 of walls, floors, roofs, suspended ceilings, stairs and partitions, or particulars of their construction and finish for the computation of dead load.

The plans should indicate the floors which are to be designed to carry partitions. Where the layout of partitions is not known, or a given layout is liable to alteration, these facts should be specially noted so that allowance may be made for partitions in any position (*see* IS : 875-1964).

e) The superimposed loads on the floors appropriate to the occupancy, as given in IS : 875-1964 or as otherwise required.

f) Details of special loads from cranes, runways, tips, lifts, bunkers, tanks, plant and equipment.

g) The grade of fire resistance appropriate to the occupancy as may be required.

G-3. INFORMATION REQUIRED BY TENDERER (IF NOT ALSO THE DESIGNER)

G-3.1 General

a) All information listed under **G-2.1**;

b) Climatic conditions at site-seasonal variations of temperature, humidity, wind velocity and direction;

c) Nature of soil. Results of the investigation of sub-soil at site of building or structure;

d) Accessibility of site and details of power supply;

e) Whether the steelwork contractor will be required to survey the site and set out or check the building or structure lines, foundations and levels;

f) Setting-out plan of foundations, stanchions and levels of bases;

g) Cross sections and elevations of the steel structure, as necessary, with large-scale details of special features;

h) Whether the connections are to be bolted, riveted or welded. Particular attention should be drawn to connections of a special nature, such as turned bolts, high strength friction grip bolts, long rivets and overhead welds;

j) Quality of steel (*see* **3**), and provisions for identification;

k) Requirements in respect of protective paintings at works and on site, galvanizing or cement wash;

m) Approximate dates for commencement and completion of erection;

n) Details of any tests which have to be made during the course of erection or upon completion; and

p) Schedule of quantities. Where the tenderer is required to take off quantities, a list should be given of the principal items to be included in the schedule.

G-3.2 Additional Information Relating to Buildings

a) Schedule of stanchions giving sizes, lengths and typical details of brackets, joints, etc;

b) Plan of grillages showing sizes, lengths and levels of grillage beams and particulars of any stiffeners required;

c) Plans of floor beams showing sizes, lengths and levels eccentricities and end moments. The beam reactions and details of the type of connection required should be shown on the plans;

d) Plan of roof steelwork. For a flat roof, the plan should give particulars similar to those of a floor plan. Where the roof is pitched, details should be given of trusses, portals, purlins, bracing, etc;

e) The steelwork drawings should preferably be to a scale of 1 to 100 and should give identification marks against all members; and

f) Particulars of holes required for services, pipes, machinery fixings, etc. Such holes should preferably be drilled at works.

G-3.3 Information Relating to Execution of Building Work

G-3.3.1 Supply of materials.

G-3.3.2 Weight of steelwork for payment.

G-3.3.3 Wastage of steel.

G-3.3.4 Insurance, freight and transport from shop to site.

G-3.3.5 Site facilities for erection.

G-3.3.6 Tools and plants.

G-3.3.7 Mode and terms of payment.

G-3.3.8 Schedules.

G-3.3.9 *Forced Majeure* — clauses and provisions for liquidation and damages for delay in completion.

G-3.3.10 Escalation clauses.

G-4. DETAILING

G-4.1 In addition to the number of copies of the approved drawings or details required under the contract, dimensioned shop drawings or details should be submitted in duplicate to the engineer who should retain one copy and return the other to the steel supplier or fabricators with his comments, if any.

G-5. TIME SCHEDULE

G-5.1 As the dates on which subsequent trades can commence, depend on the progress of erection of the steel framing, the time schedule for the latter should be carefully drawn up and agreed to by the parties concerned at a joint meeting.

G-6. PROCEDURE ON SITE

G-6.1 The steelwork contractor should be responsible for the positioning and levelling of all steelwork. Any checking or approval of the setting out by the general contractor or the engineer should not relieve the steelwork contractor of his responsibilities in this respect.

G-7. INSPECTION

G-7.0 References may be made to IS : 7215-1974, 'Indian Standard tolerances for erection of steel structures (*under preparation*)', and the 'Handbook for fabrication, erection and inspection of steel structures (*under preparation*)' for general guidance.

G-7.1 Access to Contractor's Works — The contractor should offer facilities for the inspection of the work at all stages.

G-7.2 Inspection of Fabrication — Unless otherwise agreed, the inspection should be carried out at the place of fabrication. The contractor should be responsible for the accuracy of the work and for any error which may be subsequently discovered.

G-7.3 Inspection on Site — To facilitate inspection, the contractor should during all working hours, have a foreman or properly accredited charge hand available on the site, together with a complete set of contract drawings and any further drawings and instructions which may have been issued from time to time.

G-8. MAINTENANCE

G-8.1 General — Where steelwork is to be encased in solid concrete, brickwork or masonry, the question of maintenance should not arise, but where steelwork is to be housed in hollow fire protection or is to be unprotected, particularly where the steelwork is exposed to a corroding agent, the question of painting or protective treatment of the steelwork should be given careful consideration at the construction stage, having regard to the special circumstances of the case.

G-8.2 Connections — Where connections are exposed to a corroding agent, they should be periodically inspected, and any corroded parts should be thoroughly cleaned and painted.

G-8.2.1 Where bolted connections are not solidly encased and are subject to vibratory effects of machinery or plant, they should be periodically inspected and all bolts tightened.

INDIAN STANDARDS

ON

STRUCTURAL ENGINEERING

IS:

800-1984 Code of practice for general construction in steel (*second revision*)
801-1975 Code of practice for use of cold formed light gauge steel structural members in general building construction (*first revision*)
802 Code of practice for use of structural steel in overhead transmission line towers:
(Part 1)-1977 Loads and permissible stresses (*second revision*)
(Part 2)-1978 Fabrication, galvanizing, inspection and packing
(Part 3)-1978 Testing
803-1976 Code of practice for design, fabrication and erection of vertical mild steel cylindrical welded oil storage tanks (*first revision*)
805-1968 Code of practice for use of steel in gravity water tanks
806-1968 Code of practice for use of steel tubes in general building construction (*revised*)
807-1968 Code of practice for design, manufacture, erection and testing (structural portion) of cranes and hoists (*first revision*)
1915-1961 Code of practice for steel bridges
3177-1977 Code of practice for design of overhead travelling cranes and gantry cranes other than steel-works cranes (*first revision*)
4000-1967 Code of practice for assembly of structural joints using high tensile friction grip fasteners
4014 Code of practice for steel tubular scaffoldings:
(Part 1)-1967 Definitions and materials
(Part 2)-1967 Safety regulations for scaffolding
4137-1967 Code of practice for heavy duty electric overhead travelling cranes including special service machines for use in steel works
4573-1982 Code of practice for design of mobile cranes (all types) (*first revision*)
6533-1971 Code of practice for design and construction of steel chimneys
7205-1974 Safety code for erection of structural steelwork
8147-1976 Code of practice for use of aluminium alloys in structures
8640-1977 Recommendations for dimensional parameters for industrial buildings
9178 Criteria for design of steel bins for storage of bulk materials:
(Part 1)-1979 General requirements and assessment of loads
(Part 2)-1979 Design criteria
(Part 3)-1981 Bins designed for mass flow and funnel flow
9964 Recommendations for maintenance and operation of petroleum storage tanks:
(Part 1)-1981 Preparation of tanks for safe entry and work
(Part 2)-1981 Inspection

General

804-1967 Rectangular pressed steel tanks (*first revision*)
7215-1974 Tolerances for fabrication of steel structures
8081-1976 Slotted sections
SP : 6 ISI Handbooks for Structural Engineers
SP : 6 (1)-1964 Structural steel sections
SP : 6 (2)-1962 Steel beams and plate girders
SP : 6 (3)-1962 Steel column and struts
SP : 6 (4)-1969 Use of high strength friction grip bolts
SP : 6 (5)-1980 Structural use of light gauge steel
SP : 6 (6)-1972 Application of plastic theory in design of steel structures
SP : 6 (7)-1972 Simple welded girders

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TO
IS 800 : 1984 CODE OF PRACTICE FOR GENERAL CONSTRUCTION IN STEEL

(Second Revision)

(*Page* 17, *clause* 1.4) — Substitute the following for the existing clause:

1.4 Reference

1.4.1 The following Indian Standards contain provisions which through reference in this text, constitute provision of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
456 : 1978	Code of practice for plain and reinforced concrete (*third revision*)
696 : 1972	Code of practice for general engineering drawings (*second revision*)
786 : 1967	(Supplement) SI supplement to Indian Standard conversion factors and conversion tables (*first revision*)
812 : 1957	Glossary of terms relating to welding and cutting of metals
813 : 1966	Scheme of symbols for welding
814 : 1991	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel (*fifth revision*)
816 :1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
817 : 1966	Code of practice for training and testing of metal arc welders (*revised*)

IS No.	*Title*
819 : 1957	Code of practice for resistance spot welding for light assemblies in mild steel
819 (Parts 1 and 2) : 1993	ISO system of limits and fits
875(Parts 1 to 5) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures
961 : 1975	Structural steel (high tensile) (*second revision*)
962 : 1989	Code of practice for architectural and building drawings (*second revision*)
1024 : 1979	Code of practice for use of welding in bridges and structures subject to dynamic loading (*first revision*)
1030 : 1989	Carbon steel castings for general engineering purposes (*fourth revision*)
1148 : 1982	Hot rolled steel rivet bars (up to 40 mm diameter) for structural purposes (*third revision*)
1149 : 1982	High tensile steel rivet bars for structural purposes (*third revision*)
1261 : 1959	Code of practice for seam welding in mild steel
1278 : 1972	Filler rods and wires for gas welding (*second revision*)
1323 :1982	Code of practice for oxy-acetylene welding for structural work in mild steel (*second revision*)
1363(Parts 1 to 3) : 1992	Hexagon head bolts, screws and nuts of product grade C
1364 (Parts 1 to 5) : 1992	Hexagon head bolts, screws and nuts of product grade A and B
1367 (Parts 1 to 18)	Technical supply conditions for threaded steel fasteners
1393 : 1961	Code of practice for training and testing of oxy-acetylene welders

IS No.	*Title*
1395 : 1982	Molybdenum and chromium molybdenum vanadium low alloy steel electrodes for metal arc welding (*third revision*)
1477 (Parts 1 and 2): 1971	Code of practice for painting of ferrous metals in buildings
1893 : 1984	Criteria for earthquake resistant design of structures (*fourth revision*)
1929 : 1982	Hot forged steel rivets for hot closing (12 to 36 mm diameter) (*first revision*)
1977 : 1975	Structural steel (ordinary quality) (*second revision*)
2062 :1992	Steel for general structural purposes (*fourth revision*) (*supersedes* IS 226 : 1975)
2155 : 1982	Cold forged solid steel rivets for hot closing (6 to 16 mm diameter) (*first revision*)
3613 : 1974	Acceptance tests for wire-flux combinations for submerged-arc welding of structural steels (*first revision*)
3640 : 1982	Hexagon fit bolts (*first revision*)
3757: 1985	High-strength structural bolts (*second revision*)
4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)
5369 : 1975	General requirements for plain washers and lock washers (*first revision*)
5370 : 1969	Plain washers with outside diameter 3 X inside diameter
5372 : 1975	Taper washers for channels (ISMC) (*first revision*)
5374 : 1975	Taper washers for I-beams (ISMB) (*first revision*)
6419 : 1971	Welding rods and bare electrodes for gas shielded arc welding of structural steel

IS No.	*Title*
6560 : 1972	Molybdenum and chromium-molybdenum low alloy steel welding rods and base electrodes for gas shielded arc welding
6610 : 1972	Heavy washers for steel structures
6623 : 1985	High strength structural nuts (*first revision*)
6639 : 1972	Hexagon bolts for steel structures (to be withdrawn)
6649 : 1985	Hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
7205 : 1974	Safety code for erection of structural steel work
7215 : 1974	Tolerances for fabrication of steel structures
7280 : 1974	Bare wire electrodes for submerged arc welding of structural steels
7307 (Part 1) : 1974	Approval tests for welding procedures : Part 1 Fusion welding of steel
7310 (Part 1) : 1974	Approval tests for welders working to approved welding procedures : Part 1 Fusion welding of steel
7318 (Part 1) : 1974	Approval tests for welders when welding procedure approval is not required : Part 1 Fusion welding of steel
8500 : 1991	Structural steel — Micro alloyed (medium and high strength qualities) (*first revision*)
9595 : 1980	Recommendations for metal arc welding of carbon and carbon manganese steels

NOTES 1. In lieu of IS 2062 : 1992 superseding IS 226 : 1975, replace IS 226 : 1975 by IS 2062 wherever appears in the text of the standard.

2 Wherever an Indian Standard is referred in the text, the version indicated in 1.4 shall be followed.

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