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HANDBOOK
FOR
STRUCTURAL ENGINEERS

3. STEEL COLUMNS AND STRUTS

BUREAU OF INDIAN STANDARDS

STRUCTURAL ENGINEERS' HANDBOOK No. 3

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3. STEEL COLUMNS AND STRUTS

BUREAU OF INDIAN STANDARDS
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FOREWORD

This handbook, which has been processed by the Structural Engineering Sectional Committee, SMDC 7, the composition of which is given in Appendix B, has been approved for publication by the Structural and Metals Division Council of ISI.

Steel, which is a very important basic raw material for industrialization, had been receiving considerable attention from the Planning Commission even from the very early stages of the country's First Five Year Plan period. The Planning Commission not only envisaged an increase in production capacity in the country, but also considered the question of even greater importance, namely, the taking of urgent measures for the conservation of available resources. Its expert committees came to the conclusion that a good proportion of the steel consumed by the structural steel industry in India could be saved if higher efficiency procedures were adopted in the production and use of steel. The Planning Commission, therefore, recommended to the Government of India that the Indian Standards Institution should take up a Steel Economy Project and prepare a series of Indian Standard Specifications and Codes of Practice in the field of steel production and utilization.

Over six years of continuous study in India and abroad, and the deliberations at numerous sittings of committees, panels and study groups, have resulted in the formulation of a number of Indian Standards in the field of steel production, design and use, a list of which is included in Appendix A.

The basic Indian Standards on structural steel sections are:

- IS : 808-1957 SPECIFICATION FOR ROLLED STEEL BEAM, CHANNEL AND ANGLE SECTIONS (Since revised and split up into parts)
- IS : 811-1961 SPECIFICATION FOR COLD FORMED LIGHT GAUGE STRUCTURAL STEEL SECTIONS (Since revised)
- IS : 1161-1958 SPECIFICATION FOR STEEL TUBES FOR STRUCTURAL PURPOSES (Second revision published in 1968)
- IS : 1173-1957 SPECIFICATION FOR ROLLED STEEL SECTIONS, TEE BARS (Since revised)
- IS : 1252-1958 SPECIFICATION FOR ROLLED STEEL SECTIONS, BULB ANGLES
- IS : 1730-1961 DIMENSIONS FOR STEEL PLATE, SHEET AND STRIP FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES (Since revised and split up into parts)
- IS : 1731-1961 DIMENSIONS FOR STEEL FLATS FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES (Since revised)
- IS : 1732-1961 DIMENSIONS FOR ROUND AND SQUARE STEEL BARS FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES (Since revised)

The design and fabrication of steel structures is covered by the following basic Indian Standards:

- IS : 800-1956 CODE OF PRACTICE FOR USE OF STRUCTURAL STEEL IN GENERAL BUILDING CONSTRUCTION (Since revised)

IS: 801-1958 CODE OF PRACTICE FOR USE OF COLD FORMED LIGHT GAUGE STEEL STRUCTURAL MEMBERS IN GENERAL BUILDING CONSTRUCTION (Since revised)

IS: 806-1957 CODE OF PRACTICE FOR USE OF STEEL TUBES IN GENERAL BUILDING CONSTRUCTION (Since revised)

IS: 816-1956 CODE OF PRACTICE FOR USE OF METAL ARC WELDING FOR GENERAL CONSTRUCTION IN MILD STEEL (Since revised)

IS: 819-1957 CODE OF PRACTICE FOR RESISTANCE SPOT WELDING FOR LIGHT ASSEMBLIES IN MILD STEEL

IS: 823- CODE OF PROCEDURE FOR METAL ARC WELDING OF MILD STEEL (Under preparation) (Printed in 1964)

IS: 1024- CODE OF PRACTICE FOR WELDING OF STRUCTURES SUBJECT TO DYNAMIC LOADING (Under preparation) (Printed in 1968)

IS: 1261-1959 CODE OF PRACTICE FOR SEAM WELDING IN MILD STEEL

IS: 1323-1959 CODE OF PRACTICE FOR OXY-ACETYLENE WELDING FOR STRUCTURAL WORK IN MILD STEEL (Since revised)

ISI undertook the preparation of a number of design handbooks. This handbook, which is the third in the series, relates to steel columns and struts. The first one on structural steel sections was published in March 1959. The second handbook, which deals with steel beams and plate girders, is being simultaneously published along with this handbook. Other handbooks proposed to be published in the series in due course are expected to cover the following subjects:

1) Application of plastic theory in design of steel structures
2) Designing and detailing welded joints and connections
3) Design of rigid frame structures in steel
4) Economy of steel through choice of fabrication methods
5) Functions of good design in steel economy
6) High strength bolting in steel structures
7) Large span shed type buildings in steel
8) Light-weight open web steel joist construction
9) Multi-storey steel framed structures for offices and residences
10) Roof trusses in steel
11) Single-storey industrial and mill type buildings in steel
12) Steel transmission towers
13) Steelwork in cranes and hoists
14) Structural use of light gauge sections
15) Structural use of tubular sections

Metric system has been adopted in India and all quantities, dimensions and design examples have been given in this system.

This handbook is not intended to replace text books on the subject. With this object in view, theoretical treatment has been kept to the minimum needed. Special effort has been made to introduce only modern and practical methods of analysis and design that will result in economy in utilization of steel.

The information contained in this handbook may be broadly summarized as follows:

a) Explanation of the secant formula adopted in IS : 800-1956,
b) Design examples in a format similar to that used in a design office,
c) Commentary on the design examples, and
d) Tables of important design data.

In accordance with the main objectives, those types of columns and strut designs that lead to the greatest weight saving in steel have been emphasized, as far as possible.

The calculations shown in the design examples have all been worked out using the ordinary slide rules. The metric sizes of rivets and plates incorporated in the design examples are likely to be the standard metric sizes which would be produced in this country. Indian Standards for these products are under preparation.

This handbook is based on and requires reference to the following publications issued by ISI :

IS : 226-1958 SPECIFICATION FOR STRUCTURAL STEEL (Second revision) (Fifth revision published in 1975)

IS : 800-1956 CODE OF PRACTICE FOR USE OF STRUCTURAL STEEL IN GENERAL BUILDING CONSTRUCTION (Since revised)

IS : 806-1957 CODE OF PRACTICE FOR USE OF STEEL TUBES IN GENERAL BUILDING CONSTRUCTION (Since revised)

IS : 808-1957 SPECIFICATION FOR ROLLED STEEL BEAM, CHANNEL AND ANGLE SECTIONS (Since revised and split up into parts)

IS : 816-1956 CODE OF PRACTICE FOR USE OF METAL ARC WELDING FOR GENERAL CONSTRUCTION IN MILD STEEL (Since revised)

IS : 875-1957 CODE OF PRACTICE FOR STRUCTURAL SAFETY OF BUILDINGS : LOADING STANDARDS (Since revised)

IS : 1161-1958 SPECIFICATION FOR STEEL TUBES FOR STRUCTURAL PURPOSES (Second revision published in 1968)

ISI HANDBOOK FOR STRUCTURAL ENGINEERS : 1. STRUCTURAL STEEL SECTIONS

ISI HANDBOOK FOR STRUCTURAL ENGINEERS ON SINGLE-STOREY INDUSTRIAL AND MILL TYPE BUILDINGS IN STEEL (Under preparation)

ISI HANDBOOK FOR STRUCTURAL ENGINEERS ON USE OF STEEL TUBES AS STRUCTURAL MATERIAL (Under preparation)

ISI HANDBOOK FOR STRUCTURAL ENGINEERS ON MULTI-STOREY STEEL FRAMED STRUCTURES (Under preparation)

In the preparation of this handbook, the technical committee has derived valuable assistance from Dr Bruce G. Johnston, Professor of Structural Engineering, University of Michigan, Ann Arbor. Dr Bruce G. Johnston prepared the preliminary draft of this handbook. This assistance was made available to ISI through Messrs Ramseyer & Miller, Inc, Iron & Steel Industry Consultants, New York, by the Technical Co-operation Mission to India of the Government of USA under their Technical Assistance Programme.

The photographs in this handbook have been provided through the courtesies of American Institute of Steel Construction, New York, and Butler Manufacturing Co, Kansas City, USA.

No handbook of this type can be made complete for all times to come at the very first attempt. As designers and engineers begin to use it, they will be able to suggest modifications and additions for improving its utility. They are requested to send such valuable suggestions to ISI which will be received with appreciation and gratitude.

SYMBOLS

Symbols used in this handbook shall have the meaning assigned to them as indicated below:

A	=	Area of section; Greater projection of the base plate beyond the column
a	=	Distance between the main components in a laced or battened section or width of rectangular stress block in bearing plate design
B	=	Lesser projection of the base plate beyond the column
b	=	Flange width
d	=	Depth of a section; In rivet groups, the diagonal distance between two rivets; Spacing of battens in a battened section
d_1	=	External diameter of a tube
d_2	=	Internal diameter of a tube
E	=	Young's modulus
E_t	=	Tangent modulus
e	=	Eccentricity
$\frac{ec}{r^2}$	=	Eccentricity ratio
F_1	=	Longitudinal shear
F_a	=	Permissible axial stress
F_b	=	Permissible bending stress
F_c	=	Permissible stress in direct compression
f_a	=	Calculated axial stress
f_b	=	Calculated bending stress
f_p	=	Stress at proportional limit
f_v	=	Calculated average shear stress in the section
I	=	Moment of inertia
I_{AA}	=	Moment of inertia about A-A axis
I_{BB}	=	Moment of inertia about B-B axis
I_x	=	Moment of inertia about X-X axis
I_y	=	Moment of inertia about Y-Y axis

I_{mn} = Moment of inertia of a column section between mth and nth floor levels

K = Coefficient of effective length

L = Actual length

l = Effective length ($=KL$)

l_x = Effective length about X-X axis

l_y = Effective length about Y-Y axis

l/r = Slenderness ratio

M = Bending moment

M_m = Total bending moment in the column section at mth floor level

M_{mn} = Distribution of the bending moment at the mth floor level in the column section between mth and nth floor levels

P = Axial load

P_{mn} = Axial load in the column section between mth and nth floor levels

Q = Static moment about the centroidal axis of the portion of cross-sectional area beyond the location at which the stress is being determined

R_A = Reaction at A

R_x = Component of the rivet strength in X-X direction

R_y = Component of the rivet strength in Y-Y direction

r = Radius of gyration

r_{BB} = Radius of gyration about B-B axis

r_{min} = Minimum radius of gyration

r_x = Radius of gyration about X-X axis

r_y = Radius of gyration about Y-Y axis

S = Shear

t = Thickness of base plate or splice plate; Flange or web thickness

t_f = Flange thickness

t_w = Web thickness

V = Total shear resultant on cross-section

V_t = Shear force per unit length

w = Pressure or loading on the under-side of the base plate

x	=	Distance of the rivet from a reference point along X-X axis
y	=	Distance of the rivet from a reference point along Y-Y axis
Z	=	Section modulus
Z_x	=	Section modulus about X-X axis
Z_y	=	Section modulus about Y-Y axis
Z_{mn}	=	Section modulus of the column section between *m*th and *n*th floor levels
$\triangle$	=	Deflection
℄	=	Centre line
@	=	At
$>$	=	Greater than
$<$	=	Less than
$\ngtr$	=	Not greater than
$\nless$	=	Not less than
$\simeq$	=	Approximately equal to
$\therefore$	=	Therefore

ABBREVIATIONS

Some important abbreviations used in this handbook are listed below:

Units

Area in square centimetres	cm^2
Length in centimetres	cm
Length in metres	m
Length in millimetres	mm
Load in kilograms	kg
Load in kilograms per metre	kg/m
Load in kilograms per square centimetre	kg/cm^2
Load in kilograms per square metre	kg/m^2
Load in tonnes	t
Moment in centimetre-kilograms	cm·kg
Moment in centimetre tonnes	cm·t
Moment in metre kilograms	m·kg
Moment in metre tonnes	m·t
Moment of inertia expressed in centimetre to the power of four	cm^4
Section modulus expressed in cubic centimetres	cm^3
Strength of weld in tonnes per centimetre	t/cm

Other Abbreviations

Alright	OK
Basement level	B
Centre to centre	c/c
Dead load	DL
Floor	Fl
Indian Standard Angle Section conforming to and as designated in IS : 808-1957	ISA
Indian Standard Beam Section conforming to and as designated in IS : 808-1957	ISLB, ISMB, etc
Indian Standard Channel Section conforming to and as designated in IS : 808-1957	ISLC, ISMC, etc
Live load	LL
Outside diameter	OD

SECTION I

GENERAL

1. INTRODUCTION*

1.1 A column is a structural member whose primary function is to transmit compressive force between two points in a structure. The subject of column strength has retained the interest of mathematicians and engineers alike for more than 200 years since Euler's famous contributions to column theory of 1744 and 1757.

1.2 A column is loaded and performs its useful function in compression, but, when overloaded, beyond its working strength, it does not generally fail by direct compression. Failure may be due to excessive bending or in some cases by bending combined with twisting, depending on the slenderness ratio of the compression member. If a short compression member is subjected to an axial load of sufficient magnitude, it will fail by decreasing in length and bulging, or may fail because of excessive shearing stresses if the material is brittle. If, on the other hand, a long slender strut is subjected to a relatively small axial load, the strut is in stable state and if it is displaced by a small amount due to some disturbing force, the member will straighten itself when that disturbing force is removed. For a certain increased value of the axial force, however, the member is in a state of neutral equilibrium and will remain deflected even after the removal of the disturbing force. This axial load is called the buckling load. The column will behave in the same way if, instead of a disturbing force there is a bent and/or twisted configuration existing in the member. Thus, as the length of the column increases, the cross-sectional area being constant, the load required to produce the various types of failure decreases. Therefore, columns are commonly classified as short and long columns. Even though this division may be arbitrary and there is no absolute way of determining the exact limits for each classification, for convenience of discussion in design examples of columns in this handbook this classification is being adopted.

1.3 The Euler load is the buckling load which will hold a completely elastic column in a bent position. An infinitesimal tendency to change from a straight to a bent or buckled shape will, at the Euler load cause the column so to bend. If we consider the inelastic stress-strain curve of the material, the compressive load capacity without any bending is the tangent-modulus load, Shanley having showed that if any load larger than the tangent-modulus load is applied the column will start to bend.

1.4 Thus, the tangent-modulus load provides a strength criterion for the ideally straight and centrally loaded column. In this connection, a statement published in Bulletin No. 1 of Column Research Council (of USA) may be

*Part of the introduction is abstracted from the talk on 'Basic Column Strength' presented by Dr. Bruce G. Johnston at the Fourth Technical Session of Column Research Council and published in the Proceedings of May, 1944.

quoted:

'It is quite generally accepted that the column strength may be determined with satisfactory accuracy by the use of the tangent-modulus method applied to a compressive stress-strain curve for the material, if the material throughout the cross-section of the column has reasonably uniform properties and the column does not contain appreciable residual stresses. The strength of a column may be expressed by:

$$\frac{P}{A} = \frac{\pi^2 E_T}{\left(\frac{KL}{r}\right)^2} \qquad \text{..............(1)}$$

where

$\frac{P}{A}$ = average stress in the column,

E_T = tangent modulus (slope of stress-strain curve) at stress P/A, and

$\frac{KL}{r}$ = equivalent slenderness ratio of the column.'

1.5 In the elastic range, $E_T = E$, and this substitution in equation (1) reduces it to the Euler column formula. Equation (1) may be written:

$$\frac{KL}{r} = \pi \sqrt{\frac{E_T}{P/A}} \qquad \text{..............(2)}$$

1.5.1 In equation (2), if $E_T = E$ and $P/A = f_p$ (stress at proportional limit of material), the KL/r so evaluated is the minimum slenderness ratio for which the elastic buckling occurs.

1.6 Since the failure of the column, excluding the possibility of torsion, is a matter of *bending*, one may catalogue the following two general categories of 'effects' that influence bending behaviour in real columns. These result in departure from the ideal column strength estimated by the tangent-modulus theory.

a) Accidental factors *that cause bending* in the column to take place below the tangent-modulus load:

1) Lateral loads,
2) End eccentricity, and
3) Column curvature or twist and non-homogeneity of material.

b) Factors *that modify resistance to bending*:

1) Residual stress (may increase or decrease strength);
2) Variation in inelastic stress-strain characteristics, either inherent in the material or as a result of prior tensile over-strain in all or various parts of the column.
3) Shear strength;
4) Local buckling;

5) Shape of cross-section; and

6) Lateral or end restraints (may increase strength).

1.6.1 One item has been left out of the foregoing outline, that is, compressive load, which in itself reduces bending stiffness. When an 'ideal' column buckles at the Euler load it remains perfectly straight up to that load, then, under an infinitesimal increment of load, suddenly buckles with indefinite deflections within the range wherein the assumptions inherent in the Euler derivation are valid. It would appear as if such an 'ideal' column suddenly had lost all of its bending stiffness, since the slightest touch would cause it to take any bent-position desired. This is not the case. Relatively small axial load has little effect on bending stiffness, as measured by EI, but at a gradually increasing rate, the bending stiffness reduces and as the Euler load is approached the rate of loss is quite rapid. The bending stiffness does become zero when the Euler load is reached but the variation is a continuous function of load even though the buckling itself is a discontinuous process.

1.7 If any generalization at all can be made about the list of factors that affect the strength of a column it is obvious that it is impractical to introduce them all in any mathematical way into any one column formula. On the other hand, various investigators and designers in the past have tended to over-emphasize one factor without a good enough look at the others. One is reminded of the old folk tale of the blind men, feeling various parts of an elephant, with each different man coming to a different conclusion as to what an elephant really was. The uncertainty as to what a column really is has been increased by virtue of the fact that even in laboratory tests there are usually several factors affecting column strength as determined by the testing machine. In attempting to explain any single test by a mathematical formula, it is quite possible through over-emphasis of any one factor in any particular trial 'theory', unknowingly or otherwise, to compensate for the effect of other factors that may co-exist in the tests that may be omitted from the particular theory that is on trial. Thus, one may take a given set of test data on concentrically loaded hinge-end columns and show that the test results agree with the secant formula, assuming accidental initial eccentricities of the required amount to make the theory fit the test or, on the other hand, agree with an initial curvature theory by assuming an initial curvature of the required maximum amount. Thus, there may be no proof at all that either eccentricity or curvature was the dominating factor that should have been used in the theory.

2. COLUMN DESIGN FORMULÆ AND SPECIFICATIONS

2.1 As has been stated, the tangent-modulus formula provides the most proper theoretical basis for relating the stress-strain properties of a metal to the ideal column strength of the same metal. However, for design purposes it is quite customary to determine any point on the column strength curve, especially in the case of a structural steel, as that load which will cause initial yielding in an eccentrically loaded column of that particular length. The eccentricity is arbitrarily assumed so as to give agreement between the resulting strength formula and many column tests. This is the basis for the permissible working stresses given in IS : 800-1956. The actual formula (reduced from the column strength curve by a factor of safety of 1·67) is given in Appendix D of IS : 800-1956 and is referred to in Table I of that standard. It is noted that the assumed eccentricity is in dimensionless terms:

$$\frac{ec}{r^2} = 0{\cdot}15$$

Tables I and XIII of IS : 800-1956 give permissible average stress for various l/r ratios for structural steel and high strength structural steel respectively. As noted in Appendix D of IS : 800-1956, when l/r is greater than 150, the allowable stress given by the secant formula is modified by a reduction factor which, in effect, introduces an increasing factor of safety with l/r as the value of 150 is exceeded.

2.2 To facilitate interpolation, for each integer value of l/r from 1 to 180, Table I (*see* p. 69) presents permissible stresses in agreement with **9.1.2** and Table I of IS : 800-1956, for structural steel conforming to IS : 226-1958.

2.3 The cross-sectional shape of various columns commonly used in practice is given in Table II (*see* p. 71). Also shown are approximate values of radii of gyration for these sections. In the case of the rectangular and circular sections. the values indicated are closely approximate to the correct values but for the built up section there may be a considerable fluctuation because of the variation in relative cross-sectional dimensions.

2.4 To minimize steel requirements in column design, one should keep the effective l/r as small as possible so as to use the material at the greatest possible stress. The length is given in the general design drawing and the designer should select the cross-section that will provide the largest possible radius of gyration without providing more area than is needed. Since $r = \sqrt{\frac{I}{A}}$, the largest radius of gyration is obtained when the material is farthest from the centroid. For constant area this means that the material gets thinner and thinner as the column size increases for any particular type of cross-section. This leads ultimately to such thin walls for any given column cross-section that local buckling becomes a problem and it is local buckling that ultimately limits the size to which one may go. In some cases, in order to get the material as far as possible from the neutral axis, especially when only a small load is to be carried and the total area is small, angles or channels are used together with lacing or batten plates to hold them in position as shown in Table II. The lacing bars and batten plates are not load carrying elements. They function primarily to hold the load carrying portions of the column in their relative positions and provide points of intermediate support for each separate part of the built-up column. Thus, for minimum steel requirements, batten plates and lacing bars are economical only if the increase in permissible stress for the load-carrying members permits a greater reduction in weight than is added by lacing or battens.

2.5 A column designed as centrally loaded may be accidentally loaded eccentrically or may start to bend. In such cases, there will be variable bending moments induced because of the eccentricity between the centroidal axis of the column and the resultant line of action of the applied load. As a result of the varying bending moment that is induced there will be related shearing forces in the plane of the cross-section and the lacing, batten plates, or other connecting elements should be designed to be adequate to resist this shearing force. In **21.2** of IS : 800-1956, this is arbitrarily taken as 2·5 percent of the direct load for which the column is designed. In the case of very short columns, the shearing force is induced primarily by the eccentricity of load whereas in long columns, it is primarily induced by bending. Some authorities consider that the connecting parts should be designed for the shear that would be developed when the column has finally buckled at its full load and in buckling has reached the yield point.

2.6 An important determining factor in the design of a column is the 'effective length' as influenced by end restraint conditions. There are two types of restraints, namely, position restraint or restraint against movement perpendicular to the axis of the column and direction restraint or restraint against angular rotation at the end of the column. Each type of restraint may exist about either or both axes and the conditions at the opposite ends of the column may be different. A complexity of possible combinations results but some of the more usual conditions of restraint are pictured in Appendix G (Fig. 1 to 15) of IS : 800-1956. Design examples will illustrate the use of these figures which provide interpretation of **18.1** and Table V of IS : 800-1956.

2.7 Maximum permissible slenderness ratios are given in **18.2** and Table VI of IS : 800-1956 and minimum thickness of local elements is given in terms of ratios of width to thickness in **18.4** and in Tables VI and VII of that standard.

2.8 The design of a column base slab is also covered in this Handbook as provided in **18.8** of IS : 800-1956.

2.9 Additional reductions in permissible stress for single struts or discontinuous struts are provided in **18.9** of IS: 800-1956 with allowable stresses for single angle struts given in Table X of that standard.

2.10 If bending moments are introduced into the column at axial loads below the buckling load, the column is sometimes called a 'column-in-bending' and rules for design of such members are given in **9.5** of IS : 800-1956 covering bending and axial stresses. The bending moment in a beam-column may be introduced either by lateral load or by end eccentricity and the assumed allowances for end eccentricity are given in **18.6** and Table IX of IS : 800-1956.

SECTION II

DESIGN OF CENTRALLY LOADED COLUMNS

3. INTRODUCTION

3.1 The cross-sectional shape of a centrally loaded column depends very largely on whether the column is long or short and whether it carries a small or large load. Therefore, design examples will show alternative selections suitable for the following load and length conditions:

a) Short columns with small loads,
b) Short columns with large loads,
c) Long columns with small loads, and
d) Long columns with intermediate loads.

3.2 The design examples will be discussed under the following headings pertaining to the column type rather than the length and load category:

a) Circular cross-section,
b) Single angle,
c) Double angle,
d) H-beam with welded cover plates,
e) Single cell box,
f) Laced columns, and
g) Batten plate columns.

3.3 In summary, the design problem of a centrally loaded column includes the following steps:

a) Make an initial approximation of the average allowable stress F_c;
b) Determine the required area to carry the load at the estimated allowable stress $A=P/F_c$;
c) Select a column section that will provide the estimated required area along with as large as possible a radius of gyration consistent with clearance requirements and minimum thickness limitations;
d) Calculate the radius of gyration;
e) Determine the effective slenderness ratio based on the estimated effective length according to **18.1** of IS : 800-1956;
f) Determine allowable stress from Table I as based on **9.1.2** of IS: 800-1956; and
g) Repeat steps (a) to (f), if necessary, with a revised estimate of allowable column stress.

3.4 In making the preliminary estimate of allowable stress, reference may be made to Table I with a rough approximation of the probable l/r. In the case of very short columns, or columns of any reasonable length with very heavy loads, the l/r may always be made reasonably small. In such a case the allowable stress will vary but little and a good estimate may be made at the outset.

4. SHORT COLUMNS WITH SMALL LOADS

4.1 Columns of Circular Cross-Section (*see* **Design Example 1**) — The circular cross-section may be either a solid round or a hollow cylindrical tube. Any circular cross-section has the same radius of gyration about every centroidal axis and the thin wall hollow tube provides the most effective possible disposition of material for a circular column that has the same equivalent length with respect to all axes. For a more complete discussion of tubular members, reference should be made to ISI Handbook for Structural Engineers on Use of Steel Tubes as Structural Material (under preparation).

Local buckling will not occur in the walls of a circular tube until very large ratios of radius to thickness are introduced. For practical purposes, allowing for imperfections in manufacture, it is customary to require that the tube radius be no more than about 65 times the wall thickness. Thus, for a tube having minimum permissible wall thickness of 6·3 mm the maximum radius should be about 400 mm. Minimum wall thickness permitted for tubes not exposed to weather is 3·2 mm (*see* **6.3** of IS : 806-1957).

Circular columns are especially recommended for exposed use in regions of heavy wind. The wind forces on such columns are minimized and are independent of direction.

In the following pages, designs of different types of sections used as short struts are compared for a small axial load. As a first example, tubular section is taken up for illustration. Then the other types follow. It is to be noted that the required area of cross-section for the tube is less than either the single or double angle struts designed.

4.2 Single Angle Struts (*see* **Design Example 2**) — The permissible stress in single angle struts connected by a single rivet or bolt is penalized by **18.9.1.1** of IS : 800-1956 because of the eccentricity of connection. But when connected by a weld or by two or more rivets or bolts in line along the angle at each end, the permissible stresses in accordance with Table I of this Handbook or Table I of IS : 800-1956 are applicable without any reduction, because of the end restraint effect that reduces the effect of eccentricity. The effective length l should be taken as equal to the length centre to centre of connections.

4.3 Double Angle Struts (*see* **Design Example 3**) — The double angle strut is more effective and efficient than the single angle strut, not only because of the greater permitted working stress, but also because the angles do not tend to buckle about either of their individual principal axes in respect of which the radius of gyration is the minimum. All other things being equal, if the long legs are placed back to back, the best balance of radii of gyration about the two axes of the combined section will be obtained. Attention is called to the required use of stitch rivets to ensure integral combined action of the two angles.

(*Continued on* p. 25)

Design Example 1—Short Struts (Tubular) for Small Axial Loads

Design Example 1	1* of 1
Tubular Strut	

Load $P = 10$ t

Length $l = 3$ m

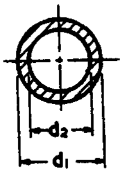
Assume permissible axial compressive stress $F_a = 1\ 000\ \text{kg/cm}^2$

$$\text{Area required } A = \frac{10\ 000}{1\ 000} = 10\ \text{cm}^2$$

Minimum wall thickness = 4 mm (*see* **6.3** of IS : 806-1957)

Try 80-mm nominal bore $\times 0{\cdot}485$ cm (*see* IS : 1161-1958)

Area $A = 12{\cdot}8\ \text{cm}^2$



$$\text{Radius of gyration} = \sqrt{\frac{d_1^2+d_2^2}{4}} = 2{\cdot}98\ \text{cm}$$

$$l/r = \frac{300}{2{\cdot}98} = 100{\cdot}5$$

Allowable $F_a = 884\ \text{kg/cm}^2$ (*see* **9.1.2** of IS : 800-1956 and Table I of this Handbook).

Allowable load $= 884 \times 12{\cdot}8 = 11\ 300$ kg or $11{\cdot}3$ t > 10 t.....OK.

* For an explanation of sheet numbering of Design Examples, *see* Footnote in Design Example 4 on p. 26.

Design Example 2—Single Angle Strut

This example indicates several trial selections leading to an angle that provides a capacity of 12·6 t. It is to be noted that one of the trial designs had to be modified because the outstanding width/thickness ratio of the angle leg was excessive. For single angle struts, the maximum permitted width/thickness ratio is 14 as compared with 16 for other outstands. This limitation is desirable because the single angle strut usually comes the nearest to torsional buckling of any rolled steel member.

Design Example 2	1 of 1
Single Angle Strut	

(Equal legs for maximum r_{min})

Assume two rivets at each end.

Allowable stresses in accordance with Table I and **18.9.1.1** (b) of IS : 800-1956

Try r for l/r $= 120 = \frac{300}{r}$; $r = 2{\cdot}5$ cm

Allowable F_c $= 709$ kg/cm² (*see* Table I of this Handbook)

Area required $= \frac{10\,000}{709} = 14{\cdot}1$ cm²

Try ISA 100 100, 8 mm.

$A = 15{\cdot}39$ cm²

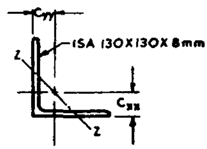
$r_{min} = 1{\cdot}95$ cm

$l/r = \frac{300}{1{\cdot}95} = 154$

Allowable F_c $= 472$ kg/cm²

Allowable load $= 472 \times 15{\cdot}39 = 7\,250$ kg—No Good.

Try ISA 130 130, 8 mm.



$A = 20{\cdot}22$ cm²

$r_{min} = 2{\cdot}55$ cm

$l/r = \frac{300}{2{\cdot}55} = 118$

Allowable F_c $= 726$ kg/cm²

Allowable load $= 726 \times 20{\cdot}22 = 14\,700$ kg — over design

Try ISA 110 110, 8 mm.

$A = 17{\cdot}02$ cm²

$r_{min} = 2{\cdot}14$ cm

$l/r = \frac{300}{2{\cdot}14} = 140$

Allowable F_c, $= 559$ kg/cm²

Allowable load $= 559 \times 17{\cdot}02 = 9\,500$ kg—No Good.

Therefore, ISA 130 130, 8 mm is the most economical section because other sections with required area and r_{min} have greater weight per metre.

$\left.\begin{array}{l}\text{Check outstanding} \\ \text{leg } \frac{130}{8}\end{array}\right\} = 16{\cdot}25 > 14$—No Good (*see* **18.4.1** of IS : 800-1956)

Effective width $= 14 \times 8 = 112$ mm

Effective area $= 17{\cdot}3$ cm² (according to **18.4.1.1** of IS : 800-1956)

Allowable F_c $=$ *726 kg/cm²

Allowable load $= 0{\cdot}726 \times 17{\cdot}3 = 12{\cdot}6$ t ......OK.

* On the basis that for computing section properties the full area of the outstanding may be taken as in a similar case for webs (*see* last sentence of **18.4.2** of IS : 800-1956).

Design Example 3 — Double Angle Strut

(Longer legs back to back connected to both sides of a 10-mm gusset by two rivets)

Design Example 3	1 of 1
Double Angle Strut	

Load and length are the same as given in Design Examples 1 and 2

Try 2 ISA 90 60, 6 mm.

$A = 2 \times 8{\cdot}65 = 17{\cdot}3\ \text{cm}^2$

$r_x = 2{\cdot}86$ cm

$r_y = 2{\cdot}55$ cm (from ISI Handbook for Structural Engineers: 1. Structural Steel Sections)

$l/r_{min} = \dfrac{300}{2{\cdot}55} = 118$—No length reduction is assumed.

Allowable F_c = 726 kg/cm² [*see* **18.9.1.2** (b) of IS: 800-1956]

Allowable load = $726 \times 17{\cdot}3 = 12\ 550$ kg—over design

Try 2 ISA 80 50, 6 mm longer legs back to back.

$A = 14{\cdot}92\ \text{cm}^2$

$r_x = 2{\cdot}54$ cm

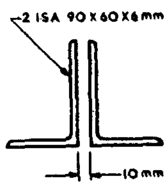
$r_y = 2{\cdot}16$ cm

$l/r_{min} = \dfrac{300}{2{\cdot}16} = 139$

Allowable F_c = 565 kg/cm²

Allowable load = $0{\cdot}565 \times 14{\cdot}92 = 8{\cdot}42$ t—No Good.

Adopt 2 ISA 90 60, 6 mm only.



r_{min} of single ISA = 1·28 cm

Maximum c/c of stitch rivets = $1{\cdot}28 \times 50 = 64$ cm (*see* **22.5** of IS: 800-1956)

(Use @ 50 cm c/c.)

Double angle struts are frequently used in single plane truss construction and it is common practice in the chords to put the short legs of unequal angles, back to back, on opposite sides of gusset plates, so as to provide the overall truss with greatest stiffness against lateral bending out of the plane of the truss.

5. SHORT COLUMNS WITH LARGE LOADS

5.1 H-Beam with Welded Cover Plate (*see* **Design Example 4**) — The H-beam by itself is a very commonly used column cross-section and the design of a number of such columns is provided later in Design Example 9 pertaining to a complete building column design. In the Design Example 4 the load is considerably greater than that in the building design example and it is necessary to add cover plates to the H-beam cross-section. This introduces the design of connecting welds as a function of required shear strength.

5.2 Single Cell Box Section (*see* **Design Example 5**) — The single cell closed box cross-section provides a very effective column, similar to the hollow tube, in that the material is disposed nearly as far as possible in all directions from the central axis and it is convenient to provide about the same radius of gyration about all axes. Although the built-up box section requires more work of fabrication, because of the longitudinal welds, it is made of plates or channels that cost less than a cylindrical tube. As in the case of the cylindrical tube, a box section is immune from torsional buckling but shall be checked as to width/thickness ratios of plate segments.

Design Example 4—Short Struts for Large Axial Loads—H-Beam with Welded Cover Plates

Design Example 4	1* of 2
Design of Cover Plates	

The load is 500 t or 50 times of that given in Design Example 1 but the length remains the same at three metres. For such a large load it is obvious that l/r will be small and a large allowable stress is assumed at the start. As soon as the basic ISHB section is selected it is possible to make a close approximation of the radius of gyration since the cover plates may be put on sufficiently wide to make the r_x minimum. Reference is made to Table II to estimate the r_y. After a selection of plates that are approximately wide enough to balance the radii of gyration about both axes, the outstanding width/thickness ratio beyond the H-beam is checked and should be less than 16. The rest of the calculations are self explanatory.

$P = 500$ t $\quad l = 3$ m

Small l/r—Trial $F_c = 1\ 200$ kg/cm²

Area required $= \dfrac{500\ 000}{1\ 200} = 417$ cm²

ISHB 450, 92·5 kg

$A = 117{\cdot}89$ cm² $\quad r_x = 18{\cdot}5$ cm $\quad r_y = 5{\cdot}08$ cm

Add plates—increase r_x and r_y to about 19

Predicted $l/r = \dfrac{300}{19} = 16$ Predicted $F_c = 1\ 228$ kg/cm²

Area required $= \dfrac{500\ 000}{1\ 228} = 407{\cdot}0$ cm²

rea supplied by ISHB 450 $= 117{\cdot}9$ cm²

Balance $= 289{\cdot}1$ cm²

$\therefore \dfrac{289{\cdot}1}{2} = 145$ cm² required per cover plate

Referring to Table II:

Approx $r_y = 0{\cdot}21\, b \quad 19 = 0{\cdot}21\, b$ (0·21 is low if plates are wide)

$\therefore$ Assume $b = \dfrac{19}{0{\cdot}25} = 76$ cm

Try **73** × 2 cm cover plate as in the sketch.

Check outstanding width: thickness ratio

$= \dfrac{24}{2} = 12$ less than 16 . . . . .OK (*see* **18.4.1** of IS: 800-1956)

Check radius of gyration

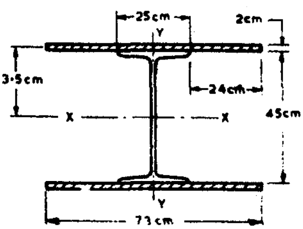
I_y—H Section $= 3\ 045$ cm⁴

I of plates $= \dfrac{4 \times 73^3}{12} = 129\ 672$ cm⁴

$\overline{132\ 717}$ cm⁴

Area $= 117{\cdot}9 + 2 \times 146 = 409{\cdot}9$ cm²

$r_y = \sqrt{\dfrac{132\ 727}{409{\cdot}9}} = 18$ cm



(r_x greater than 18·5—no need to check)

r_y is less than predicted but probably OK.

* 1 of 2 means that this Design Example has two sheets in all, of which this is the first sheet.

Welds are designed for a shear of 2·5 percent of the axial load or 12·5 t. It is to be noted that the continuous welds at each end should be as great as the maximum width of the parts jointed.	**Design Example 4**	**2 of 2**
	Design of Cover Plate Welds	

$$l/r = \frac{300}{18} = 16{\cdot}67 \quad F_e = 1\ 227\ \text{kg/cm}^2$$

$$\text{Area required} = \frac{500\ 000}{1\ 227} = 408\ \text{cm}^2$$ (near enough to 409·9 cm² provided) .....OK.

Design connecting welds for shear of 2·5 percent of P (*see* **22.2.1** of IS: 800-1956)

$$V = 0{\cdot}025 \times 500 = 12{\cdot}5\ \text{t}$$

$$\text{Shearing force per unit length, } V_t = \frac{VQ}{I}$$

$$I_x \text{ of H-Beam} = 40\ 349{\cdot}9 \simeq 40\ 350\ \text{cm}^4$$

$$I_x \text{ of plates} = 161\ 352\ \text{cm}^4$$

$$\text{Total } I_x = 201\ 702\ \text{cm}^4$$

$$Q^* = 146 \times 23{\cdot}5 = 3\ 431\ \text{cm}^3$$

$$V_t = \frac{12{\cdot}5 \times 3\ 431}{201\ 702 \times 2\dagger} = 0{\cdot}11\ \text{t/cm per weld}$$

Cover plate 2 cm thick requires minimum fillet weld of 6·0 mm (*see* Table I and **6.2.2** of IS: 816-1956)

$$\text{Shear value for weld per cm length} = 0{\cdot}6 \times 0{\cdot}70\ddagger \times 1{\cdot}025\ddagger$$

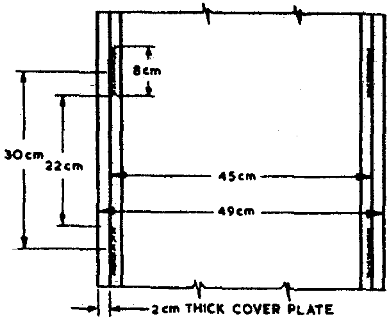
$$= 0{\cdot}43\ \text{t/cm per weld}$$

Try 0·6 × 8 cm @ 30 cm c/c intermittent.

$$\frac{8}{30} \times 0{\cdot}43 = 0{\cdot}115\ \text{t/cm per weld greater than } V_t = 0{\cdot}11\ \text{t/cm}$$ .....OK.

$$\frac{\text{Clear distance between welds}}{\text{Thickness of thinnest plate}} = \frac{220}{13{\cdot}7} = 16{\cdot}05 > 16\S$$ but may be permitted.

Use 0·6 × 8 cm @ 30 cm c/c intermittent.



* Area of cover plate to be connected multiplied by its centre of gravity distance from the neutral axis of the sections as a whole.

† Since there are two lines of welds connecting cover plate to flange of H-beam.

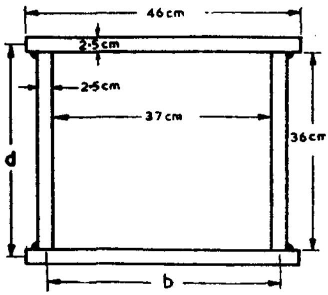
‡ *See* Table II and Table III of IS : 816-1956.

§ *See* **6.2.2** of IS : 816-1956.

Design Example 5—Short Struts for Large Axial Loads—Design with Box Section

Design Example 5	1 of 1
Design with Welded Box Section	

This provides an alternative to Design Example 4. When the box section is built up from four plates, the size may be chosen approximately in advance and the minimum radius of gyration will be approximately 0·4 of the mean minimum breadth centre to centre of plates. Thus a very accurate estimate of average allowable stress is possible in advance. The design in this particular example is direct and the width/thickness ratios are much smaller than the maximum permitted. If exposed to moist atmosphere, the box column should be hermetically sealed to minimize corrosion as there is no access to the interior of the column.



Size may be chosen approximately in advance.

Assume mean* $d = b = 36$ cm

$r_x = 0{\cdot}4\,d$

$= 14{\cdot}4$ cm (*see* Table II)

$l/r = \dfrac{300}{14{\cdot}4}$

$= 20{\cdot}8$

Allowable $F_a = 1\ 224\ \text{kg/cm}^2$

Area required $= \dfrac{500\ 000}{1\ 224}$

$= 408\ \text{cm}^2$

Trial section (increase size slightly to use 25-mm plate)

Area of 2 plates $46 \times 2{\cdot}5$ cm $= 230\ \text{cm}^2$

Area of 2 plates $36 \times 2{\cdot}5$ cm $= 180\ \text{cm}^2$

Total $= 410\ \text{cm}^2$

I_x of the two 36-cm plates $= \dfrac{5 \times 36^3}{12} = 19\ 500\ \text{cm}^4$

I_x of the two 46-cm plates $= 46 \times 5 \times 19{\cdot}25^2 = 85\ 200\ \text{cm}^4$

$104\ 700\ \text{cm}^4$

$$r = \sqrt{\frac{104\ 700}{410}} = 16\ \text{cm}$$

$$l/r = \frac{300}{16} = 19$$

Allowable $F_a = 1\ 225\ \text{kg/cm}^2$

Area required $= \dfrac{500\ 000}{1\ 225} = 408\ \text{cm}^2 =$ area provided.....OK.

Use diaphragms or caps @ each end to seal out air and hold the cross-section shape.

*** Theoretically, a square shape box section is more economical but in the present case, the load being very heavy, and the effective length small, l/r is small, therefore, for any increase in r, the effect on permissible stress is negligible and practically no economy is achieved.**

6. LONG COLUMNS WITH SMALL LOADS

6.1 Long Compression Member for Small Load (*see* **Design Example** 6) — It is generally efficient to use laced channel sections for long compression members carrying a small load. Therefore, in the design example illustrated also, it is first expected that laced channels would provide a suitable cross-section. However, a closed box section turns out to be the logical development subsequent to the initial trial of a laced channel section.

Design Example 6—Long Compression Member for a Small Load

Since the load is small and the column long, the starting point in this design is the requirement to keep the l/r ratio below the maximum permissible value of 180. The initial assumed allowable stress is that corresponding to an l/r of 180. On this basis, two ISLC 150, 14·4 kg channels are found to be satisfactory and their capacity is found at once to be greater than required. However, the flanges are too close together for fabrication to make riveted lacing bars feasible because of the insufficient clearance for backing up the rivets. ISJC *150, 9·9 kg channels are tried and found to be just sufficient.*

Design Example 6	1 of 3
Trial Design with Laced Channels	

Load $P = 10$ t; Effective length $l = 10$ m (No bracing possible)

The problem is to obtain maximum r with minimum sectional area.

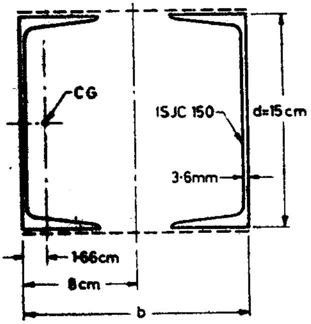
Use 2 channels or 4 angles with battens or lacing bars.

Trial Design Using Channels

Minimum depth of channel for $l/r = 180$ is determined as follows:

$$r_x = 0{\cdot}36\, d^* = r_{min}$$

$$\frac{1\ 000}{0{\cdot}36\, d} = 180;\ d = 15{\cdot}5 \text{ cm}$$



Try ISLC 150, 14·4 kg

$$A = 2 \times 18{\cdot}36 \text{ cm}^2 = 36{\cdot}72 \text{ cm}^2$$

$$r_x = 6{\cdot}16 \text{ cm}$$

NOTE — By choosing b, r_y can be made equal to r_x

$$l/r_x = \frac{1\ 000}{6{\cdot}16} = 162$$

Allowable F_c = 427 kg/cm² (*see* Table I)

Allowable load = $0{\cdot}427 \times 36{\cdot}72 = 15{\cdot}7$ t—over design

Try ISJC 150, 9·9 kg

$$A = 2 \times 12{\cdot}65 \text{ cm}^2 = 25{\cdot}3 \text{ cm}^2$$

$$r_x = 6{\cdot}16 \text{ cm}$$

$$l/r_x = \frac{1\ 000}{6{\cdot}16} = 164$$

Allowable F_c = 416 kg/cm²

Allowable load = $0{\cdot}416 \times 25{\cdot}3 = 10{\cdot}5$ t .....OK.

Approx b to make r_y = r_x (*see* **21.1.1** and **22.1.1** of IS: 800-1956)

$$0{\cdot}40\, b = 6{\cdot}16$$

$$b = 15{\cdot}4 \text{ cm} \quad \text{Adopt } b = 16 \text{ cm}$$

Check d/t of web

$$d/t = \frac{150 - 2 \times 3{\cdot}6}{3{\cdot}6} = 38 < 45$$ (*see* **18.4.2** of IS: 800-1956)

Check r_y

$$I_y = 2 \times 12{\cdot}65\,(8 - 1{\cdot}66)^2 + 2 \times 37{\cdot}9$$

$$= 1\ 090 \text{ cm}^4$$

$$r_y = \sqrt{\frac{1\ 090}{25{\cdot}3}} = 6{\cdot}55 \text{ cm} \text{ .....OK.}$$

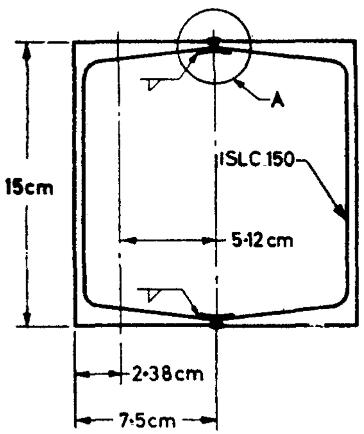
Battens or lacings are required. (These will not be designed here. **Examples of** these designs will be given under **7**.)

* *See* Table II on page 71.

Design Example 6	2 of 3
Final Design with Welded Channels as a Box	

Lacings or battens will have to be used in case the design in Sheet 1 is adopted. Returning to the original selection of two ISLC 150, it is obvious that the best way is to use these in the form of a closed box since this eliminates the necessity to use batten plates or lacing bars, which in themselves carry no load, yet add to the total steel requirement. The width/thickness ratio of the web of a compression member may go as high as 80 but only 45 t is allowed as contributing to useful load capacity. The web of the ISLC 150 is satisfactory in this respect and the details of building up channels with back-up strips to provide a satisfactory weld are shown here. The l/r is found to be just under 180 and the column capacity of the box section is about 25 percent more than that required.

As an alternative, if a solid welded box is desirable:



Try ISLC 150, 14·4 kg

$$A = 2\times 18{\cdot}36 = 36{\cdot}72 \text{ cm}^2$$

$$r_x = 6{\cdot}16$$

$$I_y = 2\times 18{\cdot}36\,(7{\cdot}5-2{\cdot}38)^2+2\times 103{\cdot}2$$

$$= 1\,169{\cdot}0 \text{ cm}^4$$

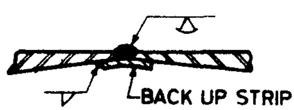
$$r_y = \sqrt{\frac{1\,169{\cdot}0}{36{\cdot}72}} = 5{\cdot}64 \text{ cm}$$

$$l/r_y = \frac{1\,000}{5{\cdot}64} = 177{\cdot}1$$

$$F_c = 350 \text{ kg/cm}^2$$

$$P = 0{\cdot}350\times 18{\cdot}36\times 2$$

$= 12{\cdot}83$ t greater than 10 t .....OK.



DETAIL AT A

Size of weld
= thickness of flange at end

$$= (7{\cdot}8-\frac{b-t_w}{2}\tan 1{\cdot}5^\circ)$$

$$= 7{\cdot}80-0{\cdot}92$$

$$= 6{\cdot}88 \text{ mm}$$

Weld to be continuous

For a long member (10 m) transmitting a small load (of, say, 9 to 10 t) as in this example, the hollow cylindrical tube is comparable to the boxed channel section.	**Design Example 6** / **Alternative Design with Tube**	3 of 3

Trial Design Using Tubes

Try IS nominal bore 150 (*see* IS: 1161-1958) 16·51 cm OD by 5·4 mm wall thickness

$$A = 27{\cdot}1 \text{ cm}^2$$

$$r = 5{\cdot}65 \text{ cm}$$

$$l/r = \frac{1\,000}{5{\cdot}65}$$

$$= 177 \text{ (Border line for } l/r \text{ of main members)}$$

$$\text{Allowable } F_a = 353 \text{ kg/cm}^2$$

$$\text{Allowable load} = 0{\cdot}353 \times 27{\cdot}1$$

$$= 9{\cdot}6 \text{ t}$$

Because of small loads, a tube or closed box is obviously economical of steel. Laced or battened column of smaller l/r would be of comparable weight because of non-stress carrying material.

7. LONG COLUMNS WITH INTERMEDIATE LOADS

7.1 Laced Columns (*see* **Design Example 7**) — For either very heavy or very light loads the use of solid box or hollow tube columns seems more economical of steel but for intermediate loads the laced or batten plate column may be selected. The lacing bars or batten plate serve to hold the load carrying portions of the column in position and shall be designed for the shear requirement as previously explained. Lacing bars are more effective than batten plates in resisting shear since they cause the column to act as a truss.

7.2 Batten Plate Columns (*see* **Design Example 8**) — It is to be noted that the batten plate column, according to **22.1.2** of IS : 800-1956, shall not be used where the compression members are subjected in the plane of the battens to eccentricity of loading.

Design Example 7—Long Compression Member for Intermediate Load

The load is 100 t and the length 11 m. A fortunate preliminary estimate of the average permissible stress as based on an estimated l/r of 92 turns out to be alright and two channels are immediately selected with a capacity just a little over the specified load of 100 t. Flanges of the channels are turned out to facilitate riveting of lacing bars. Table II provides an estimcte as to how far apart the channels should be back-to-back to balance the radii of gyration about the X-X *and* Y-Y *axes.*

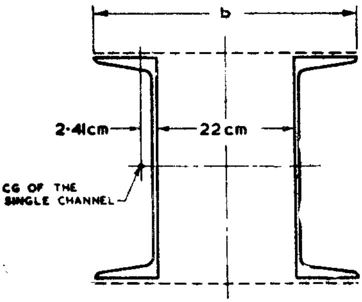
Design Example 7	1 of 2
Selection of Section	

Load $P = 100$ t; Effective length $l = 11$ m

Laced Channels — (Probably ISLC 300, 33·1 kg or ISLC 400, 45·7 kg channels required)

Preliminary estimate $r = 30{\cdot}0 \times 0{\cdot}4 = 12$ cm

$$l/r = \frac{1\,100}{12} = 92; \quad \text{Allowable } F_c = 950 \text{ kg/cm}^2$$



$$\text{Area} = \frac{100\,000}{950} = 105 \text{ cm}^2$$

Try two ISLC 350, 38·8 kg laced as shown in the sketch.

$A = 49{\cdot}47 \times 2 = 98{\cdot}94$ cm²

$r_x = 13{\cdot}72$ cm—by choosing b, r_x may be made equal to r_y

$$l/r_x = \frac{1\,100}{13{\cdot}72} = 80{\cdot}4$$

Allowable $F_c = 1\,036$ kg/cm²

Allowable load $= 1{\cdot}036 \times 98{\cdot}94$
$= 102{\cdot}5$ t .....OK.

Spacing should provide equal l/r_x and l/r_y

Assume $r_x = 0{\cdot}6\, b$ (*see* Table II)

$$b = \frac{13{\cdot}72}{0{\cdot}6} = 22{\cdot}9 \text{ cm}$$

Try $b = 22$ cm

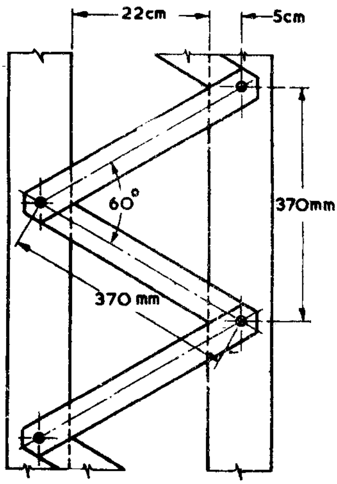
$I_y = 2 \times 49{\cdot}47\,(11{\cdot}0 + 2{\cdot}41)^2 + 2 \times 395 = 18\,581$ cm⁴

$r_y = \sqrt{18\,581/98{\cdot}94} = 13{\cdot}7$ cm

$$l/r_y = \frac{1\,100}{13{\cdot}7} = 80{\cdot}5$$

Allowable $F_c = 1\,035$ kg/cm² ..... OK.

Allowable load $= 1{\cdot}035 \times 98{\cdot}94$
$= 100{\cdot}8$ t ..... OK.



Single lacing bars @ 60° to the axis of the member.

Check l/r of channel between lacing connections.

$$l = \frac{32 \times 2}{\sqrt{3}} = 37 \text{ cm}$$

r_y of ISLC 350, 38·8 kg = 2·82 cm

$$l/r_1 = \frac{37}{2{\cdot}82} = 13{\cdot}1 < 0{\cdot}7 \times 80 \text{ ..... OK}$$

(*see* **21.6** of IS : 800-1956)

Design Example 7	2 of 2
Design of Lacings	

Here the principal design problem is the design of the lacing bars. A trial layout using flat bars is shown with an angle of 60° between successive bars. The l/r of the individual channel between lacing and connections is checked and found to be well below the maximum permissible limit.

The use of flat bars for lacing is usually suitable for very small columns but the flat bars may be changed to angles or channel sections for larger columns and various schemes, such as X bracing may be introduced to fill the requirements. A laced column using angles as lacing bars will be designed in Design Example 10 for a stepped mill building column carrying a crane load. The tensile strength and rivet values are checked and are found to be adequate. Compression strength is the controlling factor.

Tie plates are required at each end but their design is the same as for batten plates covered by alternative Design Example 8.

Lacing Bars

Minimum width = 50 mm (*see* **21.3** of IS: 800-1956)

Minimum thickness $= \frac{370}{40} = 9{\cdot}25$ mm (*see* **21.4** of IS: 800-1956)

Try 50 × 10 mm bar—r_v = 0·289 cm

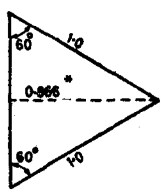
$l/r_v = \frac{37}{0{\cdot}289} = 128 < 145$ ..... OK (*see* **21.2.3** of IS: 800-1956)

F_a = 644 kg/cm²

Allowable load = 644 × 5 = 3 220 kg

Shear capacity, 2 bars (one on either side)

= 3·22 × 2 × 0·866* = 5·58 t > 2·5 percent of the load of 100 (= 2·5 t) .....OK.



Check tensile strength — 16-mm rivet
17-mm rivet hole

1 (5·0 − 1·7) 1 420 × 0·866 × 2 = 8·1 t > 2·5 t .....OK.

One 16-mm shop rivet in single shear $= 1{\cdot}025 \times \frac{1{\cdot}7^2}{4} \times \pi = 2{\cdot}32$ t

Bearing = 2 360 × 1·7 × 1 = 4 t

Single shear governs, 2·32 × 2 > 2·5 t shear capacity required ..... OK.

Supply tie plates @ each end — design is same as for batten plates in Design Example 8.

Design Example 8—Alternate Design Using Batten Plates to Replace Lacing

Design Example 8	1 of 2
Design of Batten Spacings	

The cross-section make up is the same as for the laced column in Design Example 7, hence this need not be repeated. Initially, the code provision is followed and the battens are put in with a maximum spacing between nearest rivets so as to provide an l/r of 50 maximum or 0·7 times the l/r of the member as a whole. The l/r of 50 would govern and the layout is shown. Four rivets are tried and the rivet group is checked for the moment resulting from the shear of 2·5 percent of the axial load which in this case is 2·5 t. A weight comparison shows that the batten plate column requires less total connector steel for battens than does the laced column for lacing bars.

$$\text{Shear } S = 0{\cdot}025 \times 100 = 2{\cdot}5 \text{ t}$$

$$\left.\begin{array}{l}\text{Maximum spacing}\\ \text{between nearest rivets}\end{array}\right\} = 50 \times 2{\cdot}82 = 141 \text{ cm (see \textbf{22.5} of IS: 800-1956)}$$

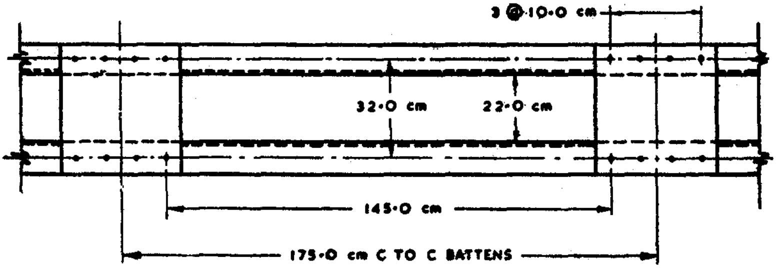
$$\text{Minimum thickness} = \frac{320}{50} = 6{\cdot}4 \text{ mm (see \textbf{22.4} of IS: 800-1956)}$$

Use 71-mm plate.

Try 4 rivets on each side.

$$\text{Rivet group } y^2 = 2\,(15^2 + 5^2) = 500 \text{ cm}^2$$

$$d = 175 \text{ cm}, \quad a = 32 \text{ cm}$$



Longitudinal shear (*see* **22.2.1.1** of IS: 800-1956):

$$F_l = \frac{2{\cdot}5 \times 175}{2 \times 32} = 6{\cdot}84 \text{ t}$$

Moment (*see* **22.2.1.1** of IS: 800-1956):

$$M = \frac{2{\cdot}5 \times 175}{2 \times 2} = 109 \text{ cm}\cdot\text{t}$$

Most stressed rivet:

$$\text{Shear} = \frac{6{\cdot}84}{4} = 1{\cdot}71 \text{ t}$$

$$\text{Bending stress} = \frac{M_n y}{\Sigma y^2} = \frac{109 \times 15}{500} = 3{\cdot}27 \text{ t}$$

$$\text{Resultant load} = \sqrt{3{\cdot}27^2 + 1{\cdot}71^2} = 3{\cdot}64 \text{ t}$$

$$\text{Bearing value of 22 mm rivet} = \frac{{}^{*}2{\cdot}3 \times 7{\cdot}1 \times 2\,360}{100 \times 1\,000}$$

$$= 3{\cdot}80 \text{ t} > 3{\cdot}64 \text{ t} \ldots\ldots \text{OK.}$$

* 22 + 1 gross diameter in mm.

In order to ensure that local failure of the main components near batten connections does not occur due to local combined stress due to bending (as a result of the 2·5 percent transverse shear in battens) and axial load, the sections are checked and the total combined fibre stress is limited to 1 575 kg/cm² by reducing the spacing of battens.

Design Example 8	2 of 2
Spacing of Battens Reduced	

Check the bending stress in the channels (though this is not particularly required in IS: 800-1956, it is considered necessary).

$$\text{Moment} = 1{\cdot}25\times 72{\cdot}5 = 90{\cdot}6\ \text{cm·t}$$

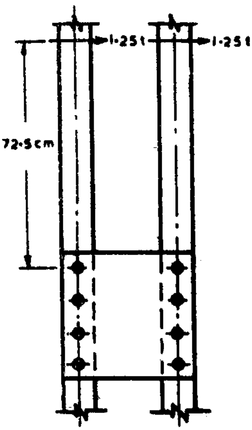
$$Z_y\ \text{(on channel)} = 52\ \text{cm}^3$$

$$f_b = \frac{1\ 250\times 72{\cdot}5}{52} = 1\ 740\ \text{kg/cm}^2$$

$$\text{Average column } f_a = 1\ 035\ \text{kg/cm}^2$$

$$\text{Combined } f_a+f_b \text{ locally} = 2\ 775\ \text{kg/cm}^2$$

NOTE — Cross-section Z_y = 52 cm³ was used.



Reduced spacing is required to allow 1 575 kg for local combined stress.

$$1\ 575 - 1\ 035 = 540\ \text{kg/cm}^2$$

is available for local bending.

The spacing required is $\frac{540}{1\ 740}\times 145 = 45$ cm

With this lower spacing adopt 2 rivets of 22 mm at 10 cm c/c instead of 4 of 22 mm at 10 cm c/c.

Revised batten spacing

Check rivet stress:

$$F_1 = \frac{2{\cdot}5\times 55}{2\times 32} = 2{\cdot}15\ \text{t}$$

$$M = \frac{2{\cdot}5\times 55}{2\times 2} = 34{\cdot}6\ \text{cm·t}$$

Most stressed rivet:

$$\text{Shear} = \frac{2{\cdot}15}{2} = 1{\cdot}1\ \text{t}$$

$$\text{Bending} = \frac{34{\cdot}6\times 5}{2\times 5^2} = 3{\cdot}46\ \text{t}$$

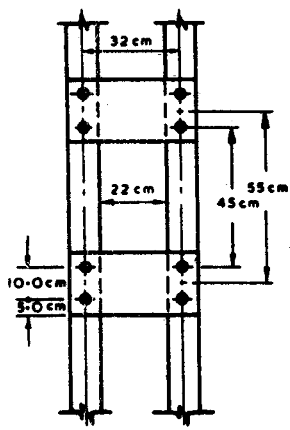
$$\text{Resultant load} = \sqrt{(3{\cdot}46)^2+(1{\cdot}1)^2}$$

$$= 3{\cdot}64\ \text{t} < 3{\cdot}8\ \text{t bearing value of 22 mm rivet.....OK.}$$

Weight comparison —laced *versus* battened

Laced—4 bars = $4\times 1\times 42\times 5$ cm³ per 37 cm of column
= 17·7 kg/m length of column
(taking density of structural steel as 0·007 85 kg/cm³)

Battened—2 plates = $2\times 20\times 42\times 0{\cdot}7$ cm³ per 55 cm of column
= 16·8 kg/m length of column



SECTION III

COLUMNS IN MULTI-STOREY BUILDINGS

8. INTRODUCTION

8.1 For a general treatment of the design of steel frames for multi-storey buildings, reference should be made to ISI Handbook for Structural Engineers on Multi-Storey Steel Framed Structures (under preparation) wherein the problem of multi-storey building column design will be treated in greater detail with reference to both vertical loads and lateral wind loads.

8.2 In the design example to follow, the details regarding distribution of load to a typical building column for dead plus live load only are given. Special design aspects related to column splices, eccentricity of floor load, and base plate design are included. Several typical building columns are shown clearly at the left side of Fig. 1. The column splices should be noted.

9. BUILDING COLUMN DESIGN FOR DEAD PLUS LIVE LOADS (*see* Design Example 9)

9.1 The building column in question will be designed for a full six-storey height of a building that includes a set-back. In the top four storeys, the column is at the exterior of the building with corresponding eccentricities of load, and in the first two and basement storeys it becomes an interior column with centric load. The basement column will be designed as a cased column.

9.2 In calculating the loads on multi-storey building columns, reference is made to IS : 875-1957. From Table I of IS : 875-1957 the loading is taken at 500 kg/m^2 of area and the imposed roof load is taken as 150 kg/m^2. Reference is also made to the reduction in imposed floor load on columns as given in **5.1** of IS : 875-1957. A uniformly distributed load of 400 kg/m^2 for weight of floors plus 100 kg/m^2 for partitions is assumed on all floors. The first floor is designed for a heavier live load of 1 000 kg/m^2 and a total dead load of 750 kg/m^2.

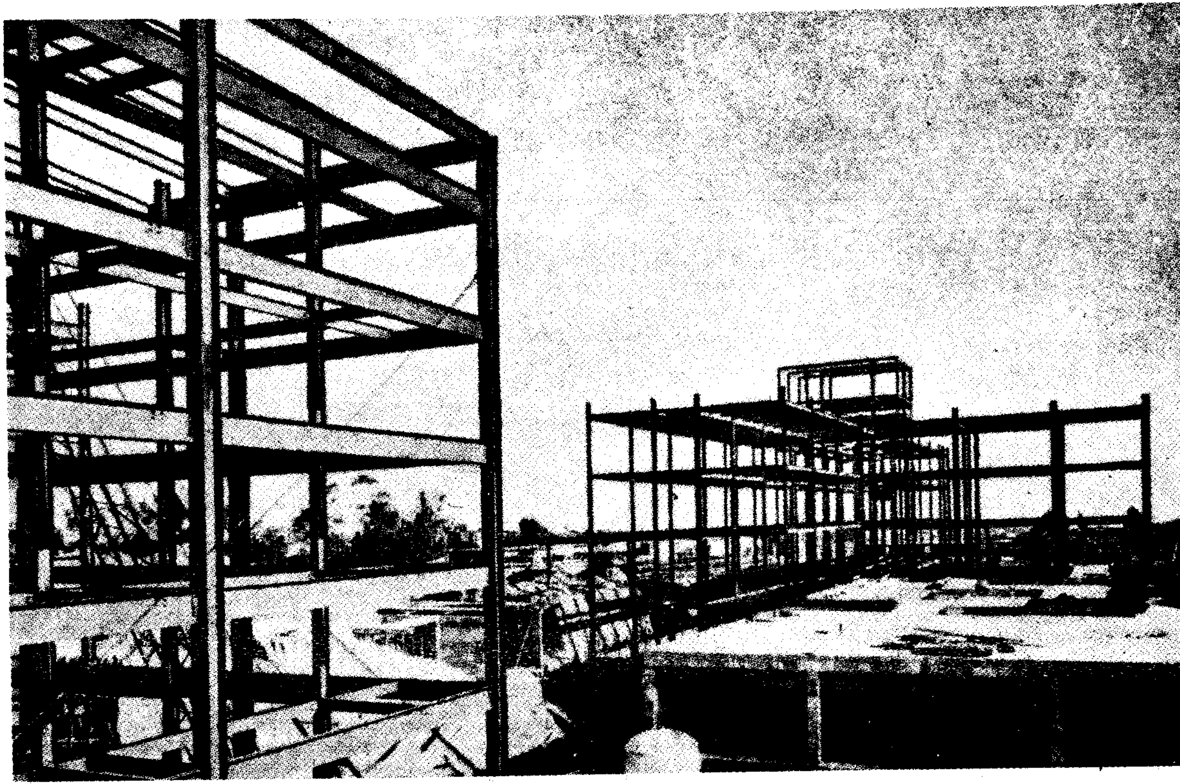


FIG. 1 TYPICAL BUILDING COLUMN IN STEEL FRAME

Photo by **A Betlehem Pacific Photo, USA**

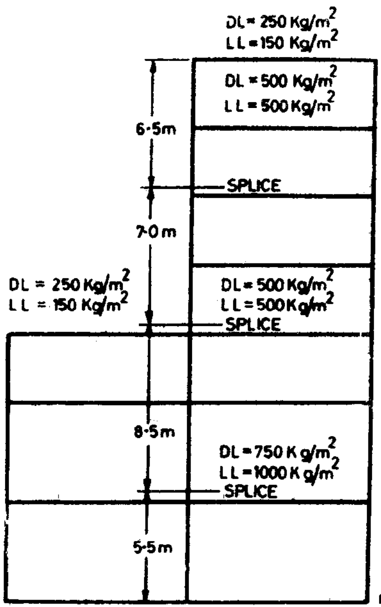
Design Example 9—Building Column Design for Dead Plus Live Loads

The portion of building pertaining to the column under design is here shown in elevation and plan, and the dead and live loads are calculated separately. Note the separation of load on each side of the column at the third floor so as to permit calculation of the eccentric moment. In the lower half of Sheet 1 are tabulated the accumulated and the reduced live loads in accordance with **5.1** *of IS : 875-1957. In the elevation, column splices are indicated 0·5 m above the first, third and fifth floor levels.*

Design Example 9	1 of 14
Details of Loads	

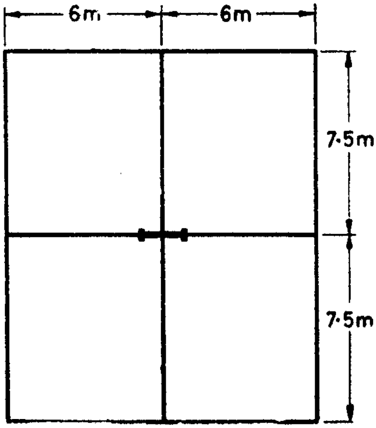
(No wind load is considered.)

Tabulation of floor Loads Coming on the Columns



	ELEVATION (m)					LOAD (Kg)
ROOF	77·50		DL	250 × 7·5×6/2	=	5 625
			LL	150 × 7·5×6/2	=	3 375
6th Fl	74·00	WALL	DL	500 × 7·5×6/2	=	18 000
			DL	900 × 7·5		
			LL	500 × 7·5×6/2	=	11 250
5th Fl	70·50	SAME AS 6th Fl	DL		=	18 000
			LL		=	11 250
4th Fl	67·00	SAME AS 6th Fl	DL		=	18 000
			LL		=	11 250
3rd Fl	63·50	LEFT SIDE	DL	250 × 7·5×6/2	=	23 625
		RIGHT SIDE	DL	500 × 7·5×6/2		
		WALL	DL	900 × 7·5		
		LEFT SIDE	LL	150 × 7·5×6/2	=	14 625
		RIGHT SIDE	LL	500 × 7·5×6/2		
2nd Fl	60·00		DL	500 × 7·5×6	=	22 500
			LL	500 × 7·5×6	=	22 500
1st Fl	55·00		DL	750 × 7·5×6	=	33 750
			LL	1000 × 7·5×6	=	45 000
BASEMENT	50·00					

Design Example 9	2 of 14
Details of Loads	



FLOOR	DL	LL	REDUCED LL	COMBINATION OF LOADS AS REDUCED [(2)+(4)]	ALTERNATE COLUMN* DESIGN LOAD
(1)	(2)	(3)	(4)	(5)	(6)
Roof	5·625	3·375	3·375	9·000	9·00
6th Fl	18·000	11·250	10·125	28·125	37·125
5th Fl	18·000	11·250	9·000	27·000	64·125
4th Fl	18·000	11·250	7·875	25·875	90·00
3rd Fl	23·625	14·625	8·775	32·400	122·40
2nd Fl	22·500	22·500	11·250	33·750	156·15
1st Fl	33·700	45·000	22·500	56·200	212·35

(All values in metric tonnes.)

* Does not include dead weight of columns.

Design Example 9	3 of 14
Column Between 5th Floor and Roof	

Since the top column runs from elevation 71·0 to 77·5, the design load is estimated at approximately midway between the fifth floor and roof with an approximate allowance of 190 kg/m for the weight of this portion of the column together with encasement. As in the case of a centrally loaded column the starting point is a trial average load but this is reduced in rough proportion to the amount of eccentricity that is expected. In the case of building columns, the calculation of eccentricity is based on **18.6** *of IS: 800-1956. At the sixth floor and at the roof, one-third of the total load is introduced with an eccentricity. This may be verified by reference to the connection details shown on Sheet 2 where it may be seen that two-thirds of the load above the set-back is introduced centrally to the column web connections and one-third comes in eccentricity through the seat angle connection to the column flange. At the sixth floor level, the eccentric moment is assumed equally divided above and below the sixth floor. It is to be noted that no reduction in live load is made in calculating the local eccentric moment.*

The column has been checked in the last sheet at the sixth floor level and there is no need to check it at the roof level since the eccentric column moment there is less than just above or below the sixth floor.

It is to be noted that in calculating the effective length of these columns, the slenderness ratio is taken as 0·67 times the slenderness ratio centre-to-centre of floors. This is in accordance with Fig. 1 of Appendix G of IS : 800-1956. Although only one beam frames into the column flange on one side, there are two beams providing direction fixity in the weak plane of bending.

Assume 3 column splices as shown in the sketch. Also note that the splices are 0·5 m above the nearest floor levels.

Top Column—5th Floor to Roof

Try F_e = 950 kg/cm²

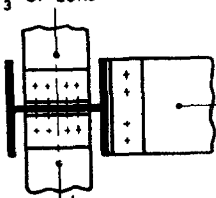
Approximate design load = 38 t (from Sheet 1)

Area required = 38 000/950 = 40 cm²

Try ISHB 150, 34·6 kg

A = 44·08 cm²

Z_x = 218·1 cm³



Top and seat connection of roof beam to column flange introduces 1/3 roof load with eccentricity as explained in the commentary above.

(*See* **18.6.1** of IS : 800-1956)

e = 7·5+2·0=9·5 cm (seat assumed to be unstiffened bracket with t=2 cm)

Moment at roof level $M_R = \dfrac{\text{Load at roof level}}{3} \times e$

$= \dfrac{9{\cdot}00}{3} \times 9{\cdot}5 = 28{\cdot}5$ cm·t

Moment at 6th floor level $M_6 = \dfrac{{}^*29{\cdot}25 \times 9{\cdot}5}{3 \times 2^\dagger} = 46{\cdot}3$ cm·t

* No reduction in live load in calculating local eccentric moment. Thus 29·25 is obtained by adding the values in 2nd and 3rd col of table of loads in Sheet 2.

† *See* **18.6.2** (b) of IS : 800-1956.

$$l/b = \frac{0{\cdot}67\times3{\cdot}5\times100}{20} = 12$$

$$F_b = 1\ 575 \text{ kg/cm}^2 \text{ (\textit{see} \textbf{9.2.2} of IS: 800-1956)}$$

$$f_b = \frac{46{\cdot}3\times1\ 000}{218{\cdot}1} = 212 \text{ kg/cm}^2$$

$$r_y = 3{\cdot}35 \text{ cm, effective}$$

$$l/r_y = \frac{0{\cdot}67\times350}{3{\cdot}35} = 70$$

$$F_c = 1\ 098 \text{ kg/cm}^2 \text{ (\textit{see} \textbf{9.1.2} of IS: 800-1956)}$$

$$\text{Axial load } P = \frac{{}^{*}190\times(3{\cdot}5+1{\cdot}75)}{1\ 000} = 38{\cdot}125 \text{ t}$$

$$f_c = \frac{38\ 125}{44{\cdot}08}$$

$$= 865 \text{ kg/cm}^2$$

$$\text{Therefore, } \frac{865}{1\ 098}+\frac{212}{1\ 575} = 0{\cdot}922<1 \text{ ..... OK.}$$

Try ISHB 150, 30·6 kg

$$A = 38{\cdot}98 \text{ cm}^2$$

$$Z_x = 205{\cdot}3 \text{ cm}^3$$

$$M_x = 46{\cdot}3 \text{ cm·t}$$

$$l/b = \frac{0{\cdot}67\times350}{20} = 12;\ F_b = 1\ 575 \text{ kg/cm}^2$$

$$f_b = \frac{46{\cdot}3\times1\ 000}{205{\cdot}3} = 225 \text{ kg/cm}^2$$

$$r_y = 3{\cdot}44 \text{ cm,} \qquad l/r_y = \frac{0{\cdot}67\times350}{3{\cdot}44} = 68$$

$$F_c = 1\ 106 \text{ kg/cm}^2$$

$$f_c = \frac{38\ 125}{38{\cdot}98} = 980 \text{ kg/cm}^2$$

$$\frac{f_c}{F_c}+\frac{f_b}{F_b} = \frac{980}{1\ 106}+\frac{225}{1\ 575}$$

$$= 0{\cdot}884+0{\cdot}143 = 1{\cdot}027>1 \text{ ..... not permitted}$$

∴ *Use ISHB 150, 34·6* kg.

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* Including 190 kg/m is for additional masonry at column up to mid height of floor (*see* commentary in Sheet 3).

Design Example 9	5 of 14
Column Between 3rd and 5th Floors	

The design of the column between the third and fifth floors splices is similar to that for the top column section as already given with the exception of the bending moment distribution at the fifth floor. According to **18.6.2** *of IS: 800-1956 if the difference in I/l is greater than 1·5, the eccentric moment is to be distributed in proportion to the I/l of the upper and lower column sections respectively. Because of the unequal distribution, the bending moment in the column at the fifth floor level is larger than at the fourth floor, but the stress condition at the fourth floor level governs the design because of the greater axial load.*

Column—3rd to 5th Floor

$$\text{Assume } f_c = 900 \text{ kg/cm}^2$$

$$\text{4th floor load} = 90{\cdot}0 \text{ t}$$

$$\text{Add weight of column} = 1{\cdot}3 \text{ t}$$

$$\text{Approximate design load} = 91{\cdot}3 \text{ t}$$

$$\text{Area required} = \frac{91{\cdot}3 \times 1\,000}{900} = 101{\cdot}5 \text{ cm}^2$$

Try ISHB 400, 77·4 kg

$$A = 98{\cdot}66 \text{ cm}^2, \qquad Z_x = 1\,404{\cdot}2 \text{ cm}^3$$

$$r_y = 5{\cdot}26 \text{ cm}, \qquad b = 25{\cdot}0 \text{ cm}$$

$$l/b = \frac{350 \times 0{\cdot}67}{25} = 9{\cdot}4, \quad F_b = 1\,575 \text{ kg/cm}^2$$

$$\text{Effective } l/r_y = \frac{350 \times 0{\cdot}67}{5{\cdot}26} = 44{\cdot}6, \; F_c = 1\,187 \text{ kg/cm}^2$$

$$M_{43} \text{ (without reduction) (assuming } t = 2{\cdot}0 \text{ cm as before)} = \frac{29{\cdot}25\,(20+2)}{3 \times 2}$$

$$= 107{\cdot}25 \text{ cm}\cdot\text{t}$$

$$P_{43} = 90{\cdot}00 + \frac{(200 \times 12{\cdot}25)^*}{1\,000} = 92{\cdot}57 \text{ t}$$

$$f_b = \frac{\dagger 107{\cdot}25 \times 1\,000}{1\,404{\cdot}2} = 76{\cdot}5 \text{ kg/cm}^2$$

$$f_c = \frac{92{\cdot}57 \times 1\,000}{98{\cdot}66} = 938 \text{ kg/cm}^2$$

$$\text{Therefore,} \quad \frac{938}{1\,187} + \frac{76{\cdot}5}{1\,575} = 0{\cdot}84 < 1 \; \ldots\ldots \text{OK.}$$

But try smaller sections:

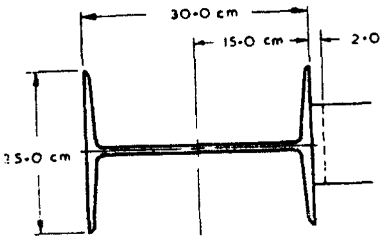
With $f_c = 1\,187$ kg/cm² as obtained in the last trial

an area of $\frac{92{\cdot}57}{1\,187} \times 1\,000 = 78$ cm² is required approximately.

* This is average weight due to column and its encasing concrete for a length of 12·25 m = 3·5 × 3 (for 4th, 5th and 6th floors) plus 3·5 × 1/2 for the 3rd floor, the section considered being midway between 3rd and 4th floor levels.

† M_{43} = 107·25 cm·t is considered and not M_{34} as it is only 45·5 cm·t as could be seen from Sheet 14.

Design Example 9	6 of 14
Column Between 3rd and 5th Floors	



Try ISHB 300, 63·0 kg

$A = 80{\cdot}25\ \text{cm}^2,\qquad r_y = 5{\cdot}29\ \text{cm}$

$Z_x = 863{\cdot}3\ \text{cm}^3,\quad \text{Effective } l/r_y = \dfrac{0{\cdot}67\times350}{5{\cdot}29} = 44{\cdot}4$

$F_c = 1\ 188\ \text{kg/cm}^2$

$M_{43} = \dfrac{29{\cdot}25\ (15+2)}{3\times2} = 82{\cdot}9\ \text{cm}\cdot\text{t}$

$F_b = 1\ 575\ \text{kg/cm}^2$

$f_b = \dfrac{82{\cdot}9\times1\ 000}{863{\cdot}3} = 96\ \text{kg/cm}^2$

$f_c = \dfrac{92{\cdot}57\times1\ 000}{80{\cdot}25} = 1\ 154\ \text{kg/cm}^2$

Therefore, $\dfrac{1\ 154}{1\ 188}+\dfrac{96}{1\ 575} = 0{\cdot}973+0{\cdot}061 = 1{\cdot}034>1$ ..... not permitted

∴ Adopt next heavier section ISHB 350, 67·4 kg

Check 4-5 section due to probable greater moments

$I_{45}/l = \dfrac{19\ 159{\cdot}7}{350} = 55{\cdot}0$

$I_{56}/l = \dfrac{1\ 635{\cdot}6}{350} = 4{\cdot}7$

The ratio between the two is greater than 1·5 (*see* **18.6.2** of IS: 800-1956)

Total moment at 5th floor

$29{\cdot}25\times\dfrac{(17{\cdot}5+2)}{3} = 190{\cdot}1\ \text{cm}\cdot\text{t}$

The distribution to the column below

$= \dfrac{55\times190{\cdot}1}{59{\cdot}7} = 175\ \text{cm}\cdot\text{t}$

$f_b = \dfrac{175\times1\ 000}{1\ 094{\cdot}8} = 160\ \text{kg/cm}^2$

f_c at 5th floor

$\dfrac{{}^{*}66\times1\ 000}{85{\cdot}91} = 768\ \text{kg/cm}^2$

$\text{Effective } l/r_y = \dfrac{350\times0{\cdot}67}{5{\cdot}34} = 44$

$F_c = 1\ 189\ \text{kg/cm}^2$

Therefore, $\dfrac{768}{1\ 189}+\dfrac{160}{1\ 575} = 0{\cdot}747<1$ ..... OK.

Use ISHB 350, 67·4 kg

* (64·125+1·9=66).

Design Example 9	7 of 14
Column Between 1st and 3rd Floors	

In designing the column between the first and the third floor splices, it is found initially that the first to second floor segment will need cover plates because the required area is greater than the area of section of any Indian Standard rolled section available. This provides an opportunity for greater steel economy and the rolled section is selected on the basis of the requirements between the second and third floor with the plan to add cover plates between the first and second floors only. The moment due to eccentricity could perhaps be maximum at the first floor level as the live load at first floor is maximum being 1 000 kg/m² and maximum eccentricity is caused when live load on one side of the floor is zero and at the other the full 1 000 kg/m² and the ratio of I/l above and below this floor is again greater than 1·5 so that the moments are proportioned accordingly. This will be checked later while finalizing the section for column 1-2 (see *Sheet 10*). *Having checked in this sheet the second to third floor segment as adequate, the additional area requirement for cover plates in the first and second floor is determined in Sheet 8.*

Column—1st to 3rd Floor

For maximum steel economy: *Try* selection for 2-3 and add cover plates in 1-2 only.

$$P_{2-3} = 122{\cdot}4 + \frac{190\times 7+210\times 7+240\ (1{\cdot}75)}{1\ 000} = 125{\cdot}6\ \text{t}$$

Assume $F_c = 1\ 100\ \text{kg/cm}^2$

$$A = \frac{125{\cdot}6\times 1\ 000}{1\ 100} = 114\ \text{cm}^2$$

Try ISHB 450, 87·2 kg

$$A = 111{\cdot}14\ \text{cm}^2$$

Calculate moment at 3rd floor level.

Refer Sheet 1.

Load calculation at 3rd floor.

Eccentric load from the left side:

DL $250\times 7{\cdot}5\times 6/2\times {}^{*}1/3 = 1\ 875$ kg

LL $150\times 7{\cdot}5\times 6/2\times 1/3 = 1\ 125$ kg

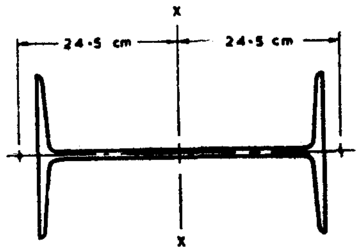
3 000 kg

From the right side:

DL $500\times 7{\cdot}5\times 6/2\times 1/3 = 3\ 750$ kg

LL $500\times 7{\cdot}5\times 6/2\times 1/3 = 3\ 750$ kg

7 500 kg



Therefore, net load causing eccentric moment:

$$7{\cdot}5-3{\cdot}0 = 4{\cdot}5\ \text{t}$$

But the worst case is when the live load is not acting on the left side on the roof.

Thus the maximum eccentric moment $M_3 = {}^{\dagger}5{\cdot}625\ (22{\cdot}5+2)$

$= 138$ cm·t

* The load causing moment about X-X axis is (as explained before) 1/3 of the total load from each side.
† $7{\cdot}5-1{\cdot}875=5{\cdot}625$ t.

Calculation of column between first and third floor is continued from Sheet 7 and the additional requirements for column between first and second floor are worked out in this sheet.	**Design Example 9** / **Column Between 1st to 3rd Floors**	**8 of 14**

As the moments of inertia of column section above and below the floor differ by more than 1·5 times the lesser, the moment due to eccentricity will be distributed in the ratio of I.

$$\text{The share of column 3-2 } \frac{I_{32}}{I_{32}+I_{34}} = \frac{39\,211}{39\,211+19\,160} = 0{\cdot}67$$

$$M_{32} = 0{\cdot}67\times 138 = 92{\cdot}5 \text{ cm}\cdot\text{t}$$

$$Z_x = 1\,742{\cdot}7 \text{ cm}^3 \text{ (of ISHB 450, 87·2 kg)}$$

$$r_y = 5{\cdot}18 \text{ cm}$$

$$\text{Effective } l/r_y = (350/5{\cdot}18)\ 0{\cdot}67 = 45{\cdot}5$$

$$F_c = 1\,184 \text{ kg/cm}^2$$

$$l/b = \frac{350}{25} = 14$$

$$F_b = 1\,575 \text{ kg/cm}^2$$

$$f_c = \frac{{}^{*}125{\cdot}6\times 1\,000}{111{\cdot}14} = 1\,130 \text{ kg/cm}^2$$

$$f_b = \frac{92{\cdot}5\times 1\,000}{1\,742{\cdot}7} = 53 \text{ kg/cm}^2$$

$$\text{Therefore, } \frac{1\,130}{1\,184} + \frac{53}{1\,575} = 0{\cdot}988 < 1 \ \ldots\ldots \text{ OK.}$$

Additional requirements between floors 1-2

Column—1st to 3rd floor

Select for axial load from 1st to 2nd floor and then check for eccentricity at 3rd floor.

$$\text{Assume } F_c = 1\,160 \text{ kg/cm}^2$$

$$P_{12} = 156{\cdot}15 + \frac{190\times 7{\cdot}0 + 210\times 7{\cdot}0 + 240\ (3{\cdot}5+5/2)}{1\,000} = 160{\cdot}4 \text{ t}$$

$$A = \frac{160{\cdot}4\times 1\,000}{1\,160} = 138 \text{ cm}^2$$

Area of ISHB 450, 87·2 kg $=$ 111·14 cm²

Area of plates required $=$ 26·86 cm²

Try 2 plates $20\times 0{\cdot}8$ cm:

$$A = 32 \text{ cm}^2$$

Calculate r_y

$$I_y\ (HB) = 2\,985{\cdot}2 \text{ cm}^4$$

$$I_y \text{ plate} = 1\,067 \text{ cm}^4$$

$$\text{Total } I_y \simeq 4\,052 \text{ cm}^4$$

$$A = 32 + 111{\cdot}14 = 143{\cdot}14 \text{ cm}^2,\ r_y = \sqrt{\frac{4\,052}{143{\cdot}14}} = 5{\cdot}33 \text{ cm}$$

$$l/r_y = \frac{0{\cdot}67\times 500}{5{\cdot}33} = 62{\cdot}8, \qquad F_c = 1\,129 \text{ kg/cm}^2$$

Capacity $= 1{\cdot}129\times 143{\cdot}14 = 161{\cdot}6 \text{ t} > 160{\cdot}4 \text{ t}$ †Tentatively .....OK.

* *See* Sheet 7.

† The moment due to eccentricity is not considered yet here in the design of section for column 1-2, as this will be done in Sheet 10.

Design Example 9	9 of 14
Column Between 1st and 2nd Floors and in Basement	

Continued from Sheet 8, the design of the cover-plated column segment between the first and second floors is similar to previous Design Example 4 for axial loads.

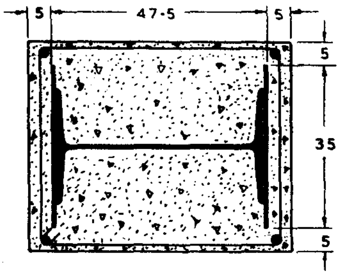
The basement section of the column is 'cased' and may be designed by a direct procedure because the width of the cover plates may be selected in advance. The radius of gyration is then determined according to **18.10** *of IS: 800-1956 on the basis of concrete encasement. Thus l/r_y is predetermined and direct calculation may be made as to the required thickness of cover plates to be added to the chosen ISHB 450, 87·2 kg.*

Stop 0·8 cm plate at 0·2 m above 2nd floor level.

Design intermittent welds same as in Design Example 4

Design Basement Section: Column cased with concrete (*see* **18.10** of IS : 800-1956)

Continue ISHB 450, 87·2 kg and use cover plate 35 cm wide.



All dimensions in centimetres.

$r_y = 0{\cdot}2\ (b+10)$ (*see* **18.10** of IS : 800-1956)

$= 0{\cdot}2 \times 45 = 9$ cm

$l/r_y = \dfrac{*0{\cdot}85 \times 500}{9} = 47{\cdot}2$

$F_c = 1\ 182$ kg/cm²

Load $P_{B1} = \dagger 218{\cdot}4$ t

. Area required $= \dfrac{218{\cdot}4 \times 1\ 000}{1\ 182} = 185$ cm²

ISHB 450, 87·2 kg;

$A = 111{\cdot}14$

Plate area required $= 73{\cdot}86$ cm²

Add cover plates $35 \times 1{\cdot}25$ each,

$A = 87{\cdot}5$ cm², Total $A = 198{\cdot}64$ cm² ..... OK.

Check moment at 1st floor level

Eccentric load from left side:

DL $750 \times 7{\cdot}5 \times 6/2 \times 1/3 = 5\ 625$ kg

LL assumed zero for maximum moment as before

Eccentric load for right side:

DL $750 \times 7{\cdot}5 \times 6/2 \times 1/3 = 5\ 625$ kg

LL $1\ 000 \times 7{\cdot}5 \times 6/2 \times 1/3 = 7\ 500$ kg

13 125 kg

Net load causing maximum moment $= 13\ 125 - 5\ 625 = 7\ 500$ kg

* Base connection will not be designed for fixing direction.

$\dagger\ 212{\cdot}35 + \dfrac{(190 \times 7{\cdot}0) + (210 \times 7{\cdot}0) + 240\ (3{\cdot}5 + \frac{5}{2}) + 360 \times 5}{1\ 000} = 218{\cdot}4.$

Design Example 9	10 of 14
Column Between Basement and 1st Floor	

Ignoring the concrete encasement

Total moment at 1st floor $= 7{\cdot}5\ (22{\cdot}5+1{\cdot}25+2) = 193$ cm· t

Moment of Inertia of basement column section about X-X axis:

$$I_{1B} = 39\ 211+87{\cdot}5\times(23{\cdot}1)^2$$
$$= 88\ 600\ \text{cm}^4$$
$$Z_{1B} = 3\ 740\ \text{cm}^3$$

Moment of Inertia of column section between the 1st and 2nd floors:

$$I_{12} = 39\ 211+32\times(22{\cdot}9)^2$$
$$= 56\ 000\ \text{cm}^4$$
$$Z_{12} = 2\ 400\ \text{cm}^3$$

Thus moments of inertia are varying by more than 1-1/2 times the lesser.

The share of column between basement and 1st floor $= \dfrac{I_{1B}}{I_{12}+I_{1B}}$

$= 0{\cdot}61$ times the total moment M_1 at 1st floor

Moment at 1st floor distributed to column between 1st and 2nd floor $= 193\times0{\cdot}39$

$= 75{\cdot}3$ cm· t

Final check of the column section between 1st and 2nd floor (continued from Sheet 8)

$$l/b = \frac{0{\cdot}67\times500}{25}$$
$$= 13{\cdot}4$$
$$F_b = 1\ 500\ \text{kg/cm}^2$$

Applying the interaction formula:

$$\frac{160{\cdot}4\times1\ 000}{143{\cdot}14\times1\ 129}+\frac{75{\cdot}3\times1\ 000}{2\ 430\times1\ 500} = 1 \not> 1\ \ldots\ldots\ \text{OK.}$$

Check the section between basement and 1st floor. In the light of **18.10.2.1** of IS: 800-1956, the steel section alone should be considered as carrying the entire load. The stiffening effect of concrete could be recognized to adopt allowable stresses of 1 500 kg/cm² in bending and 1 182 kg/cm² axial compression as determined in Sheet 9.

Moment share of basement column $= 193\times0{\cdot}6 = 115{\cdot}8$ cm·t

Therefore, $\dfrac{218{\cdot}4\times1\ 000}{195{\cdot}16\times1\ 182}+\dfrac{115{\cdot}8\times1\ 000}{3\ 660\times1\ 500} = 0{\cdot}97<1\ \ldots\ldots$ OK.

Design Example 9	11 of 14
Design of Base Plate and Splice at 5th Floor	

Design of Base Plate — *There is no particular economy (more probably, a lack of economy) in designing the foundation and column base as direction fixed. This is due to the fact that the l/r is small in any case and the permissible stress will not be greatly affected by the variation in l/r that would be induced by changing the base plate fixity. Referring to* **18.8.2** *of IS : 800-1956 the required area is obtained on the basis of 55 kg/m² bearing pressure on the concrete and the base plate thickness according to the specification formula is found to be 2·93 cm.*

(*See* **18.8.2** of IS : 800-1956)

It is assumed that the load is being distributed uniformly by the slab base.

Assume that concrete can take a bearing pressure of 55 kg/cm²:

Load = 218·4 t (*see* Sheet 9)

$$\text{Area} = \frac{218{\cdot}4 \times 1\,000}{55} = 3\,970 \text{ cm}^2$$

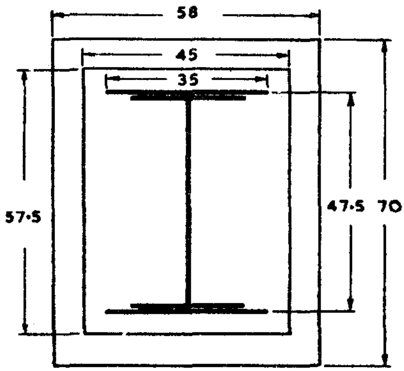
The load is assumed as distributed by the column with an area of 47·5 × 35 cm. For maximum economy in the thickness of the slab base '*t*', the projections '*A*' and '*B*' should be equal as may be seen from the formula given under **18.8.2** of IS : 800-1956.

For such equal projections, try 58 × 70 cm witn 11·5 cm and 11·4 cm projections giving an area of 4 060 cm².

$$W = \frac{218{\cdot}4 \times 1\,000}{4\,060} = 54 \text{ kg/cm}^2$$

$$t = \sqrt{\frac{3 \times 54}{1\,890}\left(11{\cdot}5^2 - \frac{11{\cdot}4^2}{4}\right)} = 2{\cdot}93 \text{ cm}$$

Use ba e plate 58 × 70 × 3 cm.



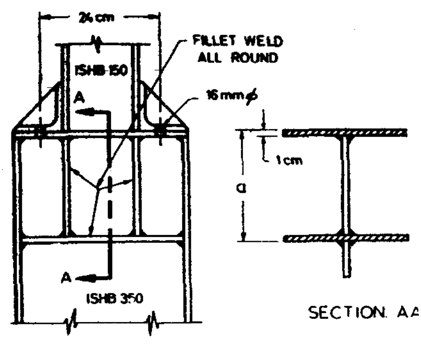
(All dimensions in centimetres.)

SPLICE AT 5TH FLOOR

The spiice is to be checked fo r two conditions, namely:

a) for moment caused by eccentricity, and
b) for axial load.

CHECK for moment capacity of the splice with details as shown in the sketch.



Assume 16 mm rivets in 17 mm rivet holes at 630 kg/cm² tension for power driven field rivet (*see* Table IV of IS : 800-1956).

Taking gauge as 45 mm for the 80 × 80 mm angle ISA 8080 used for connection, the distance between the rivets on either side is 2(4·5) + 15 = 24 cm.

$$\text{Moment capacity} = \frac{2 \times 630 \times 2{\cdot}27 \times 24^*}{1\,000} = 68{\cdot}6 \text{ cm} \cdot \text{t}$$

$$190{\cdot}1 - 175 = 15{\cdot}1 \text{ cm} \cdot \text{t (Sheet 5) ..... OK.}$$

* In this expression: 2 = number of rivets; 2·27 = area of 17-mm rivet hole; 24 = distance between rivet lines or lever arm.

Design Example 9	12 of 14
Design of Splice at 5th Floor	

At splices, if the change in column depth is small, a single bearing plate may be used to transfer the load. This is being demonstrated on Sheet 14. If there is a large change in depth, it will be more economical of steel to introduce an end detail, such as is shown on Sheet 11. In this detail, a welded WF shape is built into the top of the column. This is checked for sufficient strength in shear and bending, as if it were a short beam, to transfer a uniform distribution of stress in the column below the splice. The initial layout as shown was found to be inadequate in shear. Web doubler plates could be added, but it is simpler and more economical in the present case to deepen the beam section so as to introduce more shear capacity. Alternatively, a wedge shaped transition section could be introduced.

Check for axial load.

The axial load may be considered as being transmitted to the column section below by the specified sections acting like a short beam.

The load is assumed to be distributed uniformly at the bearing.

Total axial load at the fifth floor splice to be transmitted as detailed in Sheet 2 is 38 t.

The sections designed are ISHB 150, 34·6 kg above and ISHB 350, 67·4 kg below the splice.

The flange width = 150 mm or 15·0 cm

The depth of web between flanges $= 150-2\times9 = 132$ mm or 13·2 cm

The total length of distribution $= 15+15+13{\cdot}2 = 43{\cdot}2$ cm

Ignoring the difference in thickness between the web and flange of column section it may be assumed that the distribution of load is proportional to length and with this assumption each flange transmits:

$$\frac{38\times15}{43{\cdot}2} = 13{\cdot}2 \text{ t}$$

The load being transmitted through web $= 38-2\times13{\cdot}2 = 11{\cdot}6$ t

The flange width of the lower column section ISHB 350, 67·4 kg = 25 cm

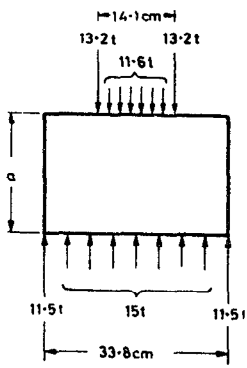
Web depth between flanges $= (35-2{\cdot}32)$

$= 32{\cdot}68$ cm

Each flange takes up $\dfrac{38\times25}{82{\cdot}68} = 11{\cdot}5$ t

Web takes up $38-2\times11{\cdot}5 = 15$ t

The loading is shown diagrammatically.



Maximum shear $= 11{\cdot}5 + \dfrac{15\times9{\cdot}9}{33{\cdot}8} = 16$ t

As sketched (section *AA*, *see* Sheet 13)

if $a =$ *say*, 14 cm, shear area $12\times0{\cdot}83^* = 9{\cdot}96$ cm²

$$f_s = \frac{16}{9{\cdot}96}\times 1\,000$$

$= 1\,610$ kg/cm² > allowable shear stress 945 kg/cm²—No Good.

* Web thickness of ISHB 350, 67·4 kg = 0·83.

Therefore, a should be increased suitably to give more shear area.

$$\frac{16\ 000}{945} \times \frac{1}{0{\cdot}83} = 20{\cdot}4 \text{ cm, } \textit{say}\text{, } 22 \text{ cm}$$

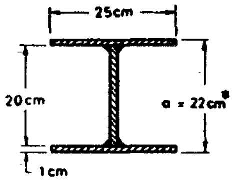
$$d/t = \frac{22}{0{\cdot}83} < 85 \text{ ..... OK (\textit{see} \textbf{20.7.1} of IS : 800-1956)}$$

Check f_b

$$\text{Moment at centre} = \frac{11{\cdot}5 \times 33{\cdot}8}{2} + \frac{15 \times 33{\cdot}8}{2 \times 4} - \frac{13{\cdot}2 \times 14{\cdot}1}{2} - \frac{11{\cdot}6 \times 14{\cdot}1}{2 \times 4}$$

$$= 144{\cdot}2 \text{ cm}\cdot\text{t}$$

The section shown in the sketch is the one resisting the moment of 144·2 cm· t.



SECTION AA

$$l/b = \frac{33{\cdot}8}{25}$$

$$= \textit{say}, 1{\cdot}5$$

$$F_b = 1\ 500 \text{ kg/cm}^2$$

$$I_x \text{ of flanges} = 2 \times 25 \times \left(\tfrac{21}{2}\right)^2$$

$$Z_x = \frac{2 \times 25 \times 21 \times 21 \times 2}{2 \times 2 \times 22}$$

$$= 503 \text{ cm}^3$$

(even ignoring web modulus)

$$f_b = \frac{144{\cdot}2 \times 1\ 000}{503}$$

$$= 287 \text{ kg/cm}^2 < 1\ 500 \text{ kg/cm}^2 \text{ ..... OK.}$$

* This was assumed as 14 cm in Sheet 11.

Design Example 9	14 of 14
Design of Splice at 3rd Floor	

Here is shown a possible detail at the third floor splice where the relative change in column size is small enough to permit use of a simple bearing plate to transfer the load. A similar splice will be required at the first floor. The bearing plate is designed as a simple beam with all of the column load conservatively estimated as being in the column flanges. The column base plate detail is shown at the bottom as previously designed at Sheet 11. Only two anchor bolts on the axis of the web are required since the column has not been assumed to be direction fixed at the base. Actually, of course, a considerable amount of direction fixity will be present, especially in view of the concrete encasement.

Design of 3rd Floor Splice

a) *Check for axial load*

Referring to Sheet 5, the column load at 3rd floor = 92·57 t

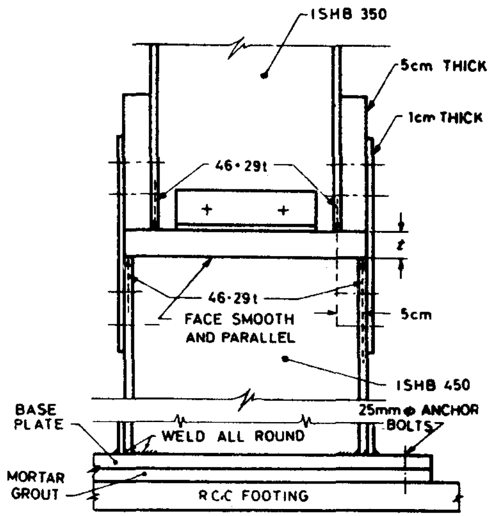
Each flange, neglecting the load taken by web, takes $\frac{92 \cdot 57}{2} = 46 \cdot 29$ t

Moment = 46·29 × 5 = 231·5 cm· t

$$t = \sqrt{\frac{231\ 500 \times 6}{1\ 890 \times 25}} = 5 \cdot 35 \text{ cm}$$

(from the usual flexure formula)

$$M = fZ = f\frac{bt^2}{6}$$



b) *Check for the moment*

The moment at the 3rd floor in column 3-4 = 138 − 92·5
= 45·5 cm· t
(*see* Sheet 8)

The rivets along the flanges shown in the sketch should be designed for a moment capacity of this 45·5 cm· t

Assuming 16 mm power driven field rivet: Shear value = 2·27 × 945 kg
= 2 140 kg

Assuming 1-cm thick splice plate: Bearing value = 1·7 × 2 125* kg
= 3 612 kg

Shear value controls

Two rivets on each side with lever arm of 45 cm have a capacity of 2·14 × 45 × 2 = 193 cm· t > 45·5 cm· t . . . . . OK.

No further extra rivets required for packing.

* *See* **10.1** of IS : 800-1956.

SECTION IV

MILL BUILDING COLUMN WITH CRANE GANTRY

10. INTRODUCTION

10.1 The stepped mill building column with crane gantry is an important design problem that combines a variety of important design questions. The column is of non-uniform cross-section, it is a 'beam-column' with both eccentric and lateral loads introduced along its length, and it involves a multiplicity of effective length questions. For the answers to matters of effective length, one is guided by Appendix G of IS : 800-1956. The column to be designed herein will be similar to that shown in Fig. 14 in Appendix G of IS : 800-1956.

10.2 There is a current practice of designing the column directly under the crane girder independently of the column that supports the building. There have been arguments and discussions over this question and it is pointed out that the assumption of separate action requires special provisions to attain it. It is recommended that the entire unit should be designed for integral action. The column section in Design Example 10 is designed with this approach in this Handbook.

11. STEPPED MILL BUILDING COLUMN WITH CRANE GANTRY (*see* Design Example 10)

11.1 It has to be understood that the example for the crane gantry column has been designed with the assumption that the top of the column is fixed in position but not in direction. Therefore, this method of design illustrated here may be followed only when these conditions are satisfied through suitable and adequate bracings at the level of the top of the column. Other examples of columns where such conditions are not satisfied will be dealt with in ISI Handbook for Structural Engineers on Single-Storey Industrial and Mill Type Buildings in Steel (under preparation). Reference should, therefore, be made to this Handbook for details and fuller discussion of the problem.

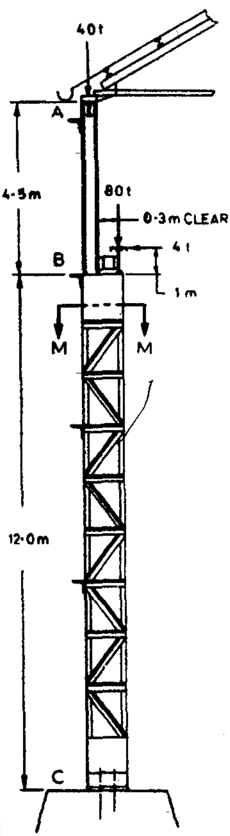
11.2 In comparison with a design based on completely separate action of crane and building column components, the consideration of the entire column as a single unit with eccentric and lateral loads will result in heavier design above the crane gantry and possibly somewhat lighter design below. A certain amount of rigidity is desirable in a mill building because of the undesirable sway and vibration that may be induced by the operation of the travelling bridge crane. It is learnt that some mill buildings in use in USA have had to undergo extensive revisions with costly additions of steel because they were too flexible with regard to side sway in the upper column segments above the crane runway girder.

Design Example 10—Stepped Mill Building Column with Crane Gantry

Cross-sectional elevation at one of the columns shows the general arrangement and dimensions. The mill building bents are assumed to be 9 m c/c and since the column size is not known at the outset it is necessary to get some preliminary estimate as to bending moments in order to approach the final design through a series of trials. The lateral load is specified as 10 percent of the crane runway reaction of 80 t and this is apportioned half to each column. This sheet shows the rough initial 'guess' as to R_A leading to an initial approximation of bending moment in the top column segment AB for which the actual load is the dead weight of the roof truss system plus superimposed load, all estimated at 40 t.

Design Example 10	1 of 12
Trial Design of Top Segment	

Stepped Mill Building Column with Crane Gantry



For effective lengths, *see* Fig. 14 of Appendix G of IS: 800-1956

Column segment A to B

To make preliminary selection, estimate bending moments:

$$\text{Estimate } R_A = 1/2 \times 4 = 2\ \text{t}$$

$$\text{Trial } M_{BA} = 2 \times 4{\cdot}5 = 9\ \text{m·t}$$

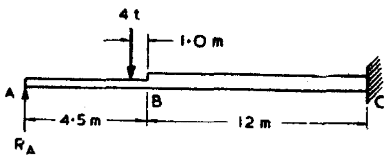
Effective length considering X_1-X_1 axis $= 1{\cdot}5\,L$

$= 1{\cdot}5 \times 4{\cdot}5$

$= 6{\cdot}75$ m (l_{x1})

Effective length considering Y_1-Y_1 axis $= 1{\cdot}0\,L$

$= 4{\cdot}5$ m (l_{y1})



Try **ISHB 300, 63·0 kg**

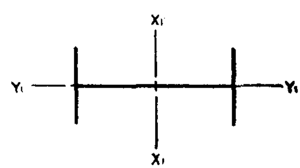
$Z_{x1} = 863{\cdot}3\ \text{cm}^3$, $A = 80{\cdot}25\ \text{cm}^2$

$r_{x1} = 12{\cdot}7$ cm, $r_{y1} = 5{\cdot}29$ cm

$$\frac{l_{x1}}{r_{x1}} = \frac{6{\cdot}75 \times 100}{12{\cdot}7} = 53$$

$$\frac{l_{y1}}{r_{y1}} = \frac{4{\cdot}5 \times 100}{5{\cdot}29} = 85$$

(This is maximum slenderness ratio.)



$$\text{Trial } f_c = \frac{40 \times 10^3}{80{\cdot}25} = 498\ \text{kg/cm}^2$$

$$f_b = \frac{9 \times 100 \times 10^3}{863{\cdot}3} = 1\,040\ \text{kg/cm}^2$$

For *Max* $l/r = l_y/r_y = 85$

$F_c = 1\,003\ \text{kg/cm}^2$

The trial selection is checked for its adequacy. It is found that the selection is slightly under-designed and since the moments are only known to a rough degree of approximation, the trial of the next heavier ISHB is suggested.	**Design Example 10** / **2nd Trial Design of Top Segment**	**2 of 12**

For determining maximum allowable bending stress for bending of the column about X_1-X_1, l_{y1}/b is to be considered as the Beam-Column section is likely to buckle laterally about Y_1-Y_1.

$$l/b = \frac{450}{25}$$

$$= 18$$

$$F_b = 1\,575 \text{ kg/cm}^2$$

$$\frac{498}{1\,003} + \frac{1\,040}{1\,575} = 0{\cdot}495 + 0{\cdot}66$$

$$= 1{\cdot}155 > 1 \text{ — No Good}$$

(*see* **9.5** of IS : 800-1956)

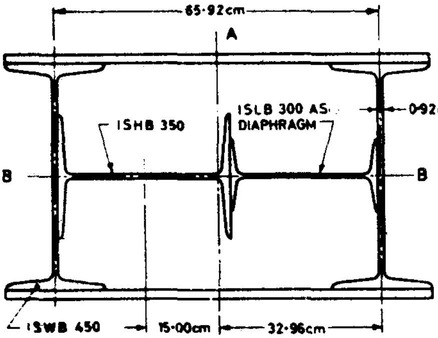
Moment due to eccentricity has been neglected.

Improve trial section for AB by adding trial eccentric moment.

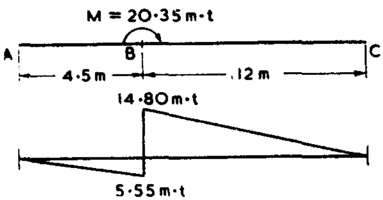
Try ISHB 350, 72·4 kg.

Trial Design of Bottom Segment

The first trial selection for the lower part of the column is shown in cross-section and the moment of inertia calculated about A-A axis. On the basis of the trial section, it is possible to estimate the eccentric moment and this is done initially as if the top and bottom at A and C were pinned. This is still only a rough approximation of the actual moments which are later determined by the moment distribution procedure on Sheet 7.



SECTION MM



Before trying ISHB 350, 72·4 kg for the upper column section—consider

Column B-C

Trial section:

Vertical load $= 80+40=120$ t

Assume $F_c = 650$ kg/cm^2

Area required $= \dfrac{120\times10^3}{650}$

$= 185$ cm^2 (ignoring eccentricity)

Try 2—ISWB 450, 79·4 kg

$A = 2\times101{\cdot}15=202{\cdot}3$ cm^2

Try the arrangement as shown in the sketch *M-M*.

Trial section M-M:

Calculate I_{AA}

$$I_{AA} = 2\times1\,706{\cdot}7 + 101{\cdot}15\,(32{\cdot}96)^2\times2$$
$$= 3\,413{\cdot}4+220\,000$$
$$= 223\,413 \text{ cm}^4$$
$$I_{BB} = 2\times35\,057{\cdot}4$$
$$= 70\,105 \text{ cm}^4$$

Applied eccentric moment $= 80\times32{\cdot}96-40\times15$
$= 2\,035$ cm·t

Preliminary approximation assuming '*C*' as pinned

$$R_A = \frac{2\,035}{16{\cdot}5\times10^2} = 1{\cdot}23 \text{ t}$$

$$M_{BA} = \frac{2\,035\times4{\cdot}5}{16{\cdot}5\times100} = 5{\cdot}55 \text{ m·t}$$ (This distribution is approximate assuming that the Section from *A* to *C* is uniform.)

$$M_{BC} = \frac{2\,035\times12}{16{\cdot}5\times100} = 14{\cdot}80 \text{ m·t}$$

Check revised selection suggested in Sheet 1 for *AB* for resisting these approximate moments also.

ISHB 350, 72·4 kg

$A = 92{\cdot}21$ cm^2

$r_x = 14{\cdot}65$ cm

$r_y = 5{\cdot}22$ cm

$Z_x = 1\,131{\cdot}6$ cm^3

Max $l/r = l/r_y = \dfrac{450}{5{\cdot}22} = 86$

$F_c = 995$ kg/cm^2

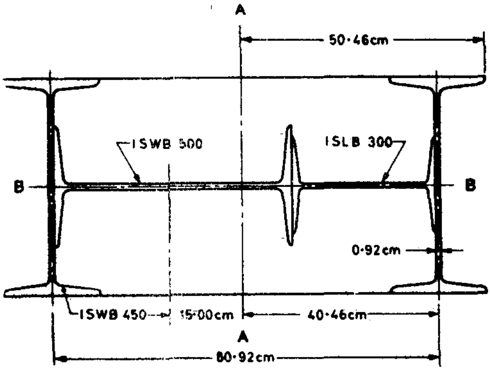
$l/b = \dfrac{450}{25} = 18$

$F_b = 1\,575$ kg/cm^2

$f_c = \dfrac{40\times10^3}{92{\cdot}21} = 434$ kg/cm^2

Design Example 10	4 of 12
Trial Design of Bottom Segment	

The initial trial design with ISHB 350, 72·4 kg is found too small and in the second trial ISWB 500, 95·2 kg sections are used. This is found to be satisfactory, still on the basis of very approximate moment estimates, the section properties in the main segment BC are determined. On this sheet also, for the first time, the additional direct force due to dead weight of walls, girts, siding and column is estimated and added to the axial load.



Total $M_{BA} = 9{\cdot}0^{*} + 5{\cdot}55^{\dagger}$

$= 14{\cdot}55$ m· t

$$f_b = \frac{14{\cdot}55 \times 10^5}{1\,131{\cdot}6} = 1\,287 \text{ kg/cm}^2$$

$$\frac{434}{995} + \frac{1\,287}{1\,575} = 0{\cdot}437 + 0{\cdot}817$$

$= 1{\cdot}254 > 1$—No Good.

Try ISWB 500, 95·2 kg

$A = 121{\cdot}22$ cm²

$Z_x = 2\,091{\cdot}6$ cm³, $r_y = 4{\cdot}96$ cm

$$Max\ l/r = \frac{450}{4{\cdot}96} = 91$$

$F_a = 958$ kg/cm², $r_x = 20{\cdot}77$ cm

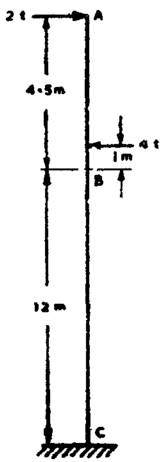
$$l/b = \frac{450}{25} = 18$$

$F_b = 1\,575$ kg/cm²

Assuming revised section *M-M* as in the sketch.

Applied eccentric moment is $80 \times 40{\cdot}46 - 40 \times 15 = 2\,637$ cm· t

Approximate moment to $AB = 2\,637 \times \dfrac{5{\cdot}5}{20{\cdot}35} = 714$ cm· t (*see* Sheet 3)



$$f_a = \frac{40 \times 10^3}{121{\cdot}222} = 331 \text{ kg/cm}^2$$

$$f_b = \frac{(900 + 714) \times 10^3}{2\,091{\cdot}6} = 775 \text{ kg/cm}^2$$

$$\frac{331}{958} + \frac{775}{1\,575} = 0{\cdot}346 + 0{\cdot}49 = 0{\cdot}84 < 1 \ldots\ldots \text{OK.}$$

Use ISWB 500, 95·2 kg *for section AB.*

Check stress in BC (see *Sheet 1*)

Due to 4 tonnes lateral load

Approx moment at $C = \pm\ (2 \times 16{\cdot}5 - 4 \times 13)$	$= \pm 19$ m· t
Approx moment at $B = \pm (2 \times 4{\cdot}5 - 4 \times 1)$	$= \pm\ 5$ m· t
Approximate moment	
at B due to vertical load $= 26{\cdot}37 - 7{\cdot}14$	$= 19{\cdot}23$ m· t
Total *Max* moment at B	$= 24{\cdot}23$ m· t
$I_{AA} = 2 \times 1\,706{\cdot}7 + 2 \times 101{\cdot}15 \times 40{\cdot}46^2$	$= 334\,600$ cm⁴
$I_{BB} = 2 \times 35\,057{\cdot}4$	$= 70\,115$ cm⁴
Estimate additional dead weight:	
at middle of segment BC—assume column spacing of	
8·5 m girts + siding @ 25 kg/m² $= 0{\cdot}025 \times 8{\cdot}5 \times 10{\cdot}5$	$= 2{\cdot}24$ t
column AB @ 95·2 kg/m $= 0\,095\,2 \times 5$	$= 0{\cdot}476$ t
column BC @ 200 kg/m (*say*) $= 0{\cdot}200 \times 6$	$= 1{\cdot}200$ t
Total	$\simeq 4{\cdot}0$ t

* *See* Sheet 1. † *See* Sheet 3.

Continuing the analysis on Sheet 4, it is found that the main section now appears over-designed and a smaller section is tried. The calculations are a repetition of the previous sheet and the smaller section is found to be satisfactory.	**Design Example 10** — **Trial Design of Bottom Segment**	**5 of 12**

$$f_c = \frac{40+80+4}{202 \cdot 3} \times 10^3 = 615 \text{ kg/cm}^2$$

$$f_b = \frac{2\,423 \times 50 \cdot 46}{334\,600} \times 10^3 = 364 \text{ kg/cm}^2, \quad r_{BB} = 18 \cdot 63 \text{ cm}$$

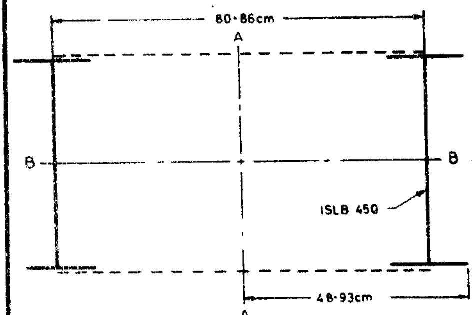
Effective $l/r = \dfrac{0 \cdot 85 \times 1\,200}{18 \cdot 63} = 55$ (*see* Fig. 14 in Appendix G of IS: 800-1956)

$$\left. \begin{aligned} F_c &= 1\,159 \text{ kg/cm}^2 \\ F_b &= 1\,500 \text{ kg/cm}^2 \end{aligned} \right\} \text{ from IS: 800-1956}$$

$$\frac{615}{1\,159} + \frac{364}{1\,500} = 0 \cdot 529 + 0 \cdot 243 = 0 \cdot 772 < 1\text{—over design}$$

Try smaller sections 2—*ISLB* **450**, 65·3 kg

Check width/thickness ratio of web (*see* **18.4.2** of IS: 800-1956)



$$d - 2t_f = 450 - 2 \times 13 \cdot 4 = 423 \cdot 2 \text{ mm}$$

$$t_w = 8 \cdot 6 \text{ mm}$$

$$d/t_w = \frac{423 \cdot 2}{8 \cdot 6} = 49 > 45$$

$\therefore$ Effective width $= 45\,t_w = 38 \cdot 7$ cm

Width reduction $= 423 - 387 = 36$ mm

Area reduction $= 3 \cdot 6 \times 0 \cdot 86 = 3 \cdot 09 \text{ cm}^2$

Effective area of section $= 83 \cdot 14 - 3 \cdot 09 = 80 \cdot 05 \text{ cm}^2$

Total $= 2 \times 80 \cdot 05 = 160 \cdot 1 \text{ cm}^2$

$$I_{AA} = 2 \times 853 + 2 \times 83 \cdot 14^* \times 40 \cdot 43^2 = 273\,500 \text{ cm}^4$$

$$I_{BB} = 2 \times 27\,536 = 55\,072 \text{ cm}^4$$

$$r_{BB} = 18 \cdot 20 \text{ cm}, \quad \text{Effective } l/r = \frac{0 \cdot 85 \times 1\,200}{18 \cdot 20} = 56$$

$$F_c = 1\,156 \text{ kg/cm}^2$$

$$F_b = 1\,500 \text{ kg/cm}^2$$

$$f_c = \frac{124 \times 10^3}{160 \cdot 1} = 775 \text{ kg/cm}^2$$

$$f_b = \frac{2\,423 \times 48 \cdot 93}{273\,500} \times 10^3 = 435 \text{ kg/cm}^2$$

$$\frac{775}{1\,156} + \frac{435}{1\,500} = 0 \cdot 67 + 0 \cdot 29 = 0 \cdot 96 < 1 \text{ ..... OK.}$$

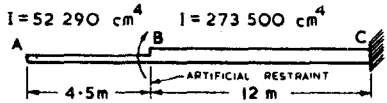
Now make accurate check on moments using moment distribution method considering *AB—BC* as separate members.

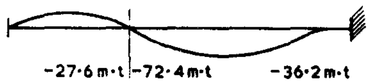
*** Full area in calculation of section properties.**

Design Example 10	6 of 12
Analysis of Forces and Moments	

A satisfactory design having been arrived at on the basis of approximate moments, these moments are now calculated more exactly. The Hardy Cross Method of moment distribution is used. It is desired to determine the bending moments in the column for an arbitrary moment introduced at B; also, for an arbitrary lateral force introduced at B. By keeping these separate it will be possible to handle combinations of load more readily. In the initial analysis for moment introduced at B, an artificial imaginary restraint is provided to hold B against lateral movement. On the basis of the resulting moments caused by an equal and opposite restraining force and superposing it on the initial solution, the effect of restraint is removed and the desired solution is obtained. The analysis for lateral force at B is started by assuming a displacement at B with no rotation. Rotation is then permitted and after distribution of moments, the force consistent with these moments is determined. Then, by proportion, the moments for unit force at B may be evaluated. Finally, there are summarized the bending moments due to a unit lateral force at B and due to a hundred units of moment at B. Now, referring back to Sheet 4, the actual moments caused by the eccentric moment and lateral force are evaluated and the combined maximum moment is given at the bottom of the next sheet.

Analysis for eccentric load—Apply unbalanced
(−) moment of 100 m· t at B





Distribution factors at B:

Stiffness of AB (one end being hinged) $= \frac{3}{4} \times \frac{52\ 290}{450} = 87$ For AB $= \frac{87}{87+228} = 0{\cdot}276$

$BC = \frac{273\ 500}{1\ 200} = 228$ For BC $= 0{\cdot}724$

Assuming restraint at B, a total applied moment of -100 m· t is distributed as

$$\left.\begin{aligned} M_{BA} &= -27{\cdot}6 \text{ m· t} \\ M_{BC} &= -72{\cdot}4 \text{ m· t} \\ \text{and } M_{CB} &= -36{\cdot}2 \text{ m· t} \end{aligned}\right\} \quad \ldots\ldots(\text{i})$$

Shear in $AB = \frac{27{\cdot}6}{4{\cdot}5} = 6{\cdot}2$ t $\downarrow\uparrow$

Shear in $BC = \frac{72{\cdot}4+36{\cdot}2}{12} = 9{\cdot}1$ t $\downarrow\uparrow$

Applied restraint $= 9{\cdot}1 - 6{\cdot}2 = 2{\cdot}9$ t $\downarrow$ at B $\quad\ldots\ldots$(ii)

Analysis for displacement with no rotation:

$$\left.\begin{aligned} M_{BA} &= \frac{3\ EI\triangle}{l^2_{AB}} = \frac{3 \times 52\ 290 E\triangle}{450^2} = 0{\cdot}775\ E\triangle \\ M_{BC} &= \frac{6\ EI\triangle}{l^2_{BC}} = \frac{6 \times 273\ 700 E\triangle}{1\ 200^2} = 1{\cdot}140\ E\triangle \end{aligned}\right\} \quad \ldots\ldots(\text{iii})$$

Based on values at relation (iii) in Sheet 6, relationship between restraining force at B and the moments in portions BA, BC and CB may be worked out as given in this sheet.	**Design Example 10** / **Analysis of Forces and Moments**	**7 of 12**

	A	B: 0·276	B: 0·724	C
Fixed end moments (FEM)		−77·5	+114	+114
Distribution		−10·1	− 26·4	
Carry over				− 13·2
Final moments		−87·6 ↺	↺ +87·6	+100·8
	Shear $= \frac{87\cdot6}{4\cdot5}$ $= 19\cdot5$ t	↓ ↑	↑↓ Shear $= \frac{87\cdot6+100\cdot8}{12}$ $= 15\cdot7$ t	
Moment distribution for 1 unit of force at *B* ↑	$\frac{87\cdot6}{19\cdot5+15\cdot7} = -2\cdot5$	−2·5	$\frac{+87\cdot6}{19\cdot5+15\cdot7}$ $= +2\cdot5$	$\frac{+100\cdot8}{19\cdot5+15\cdot7}$ $= +2\cdot87$

..........(iv)

Based on this result, the relationship between the applied moment at *B* and final distributed moments due to the applied moment without any artificial restraint at *B* for lateral movement may be worked out.

Applied moment −100 m· t

	A	B	B	C
a) Distribution with restraint at *B* (*see* Sheet 6)		+27·6	+72·4	+36·2
b) For releasing the restraint of 2·9 t ↓ (*see* Sheet 6) from the relation (iv)		+7·2	− 7·2	−8·3
Final distribution for 100 units of moment at *B*		+34·8	+65·2	+27·9

..........(v)

From these results, the final distribution of moments in the problem under design here could be worked out.

Applied loads are:

a) lateral load of ±4 t at *B* (*see* Sheet 4)
b) moment of ±4 m· t at *B*
c) moment due to eccentricity = −26·4 m· t (*see* Sheet 4)

	A	B	B	C
For ±4 t lateral load at *B*		∓10	±10 (from iv)	±11·48
For ±4 m· t at *B*		∓ 1·4	∓ 2·6 (from v)	∓ 1·12
For −26·4 m· t (due to eccentricity)		+ 9·2	+17·2 (from v)	+ 7·1
Maximum combined moment		+20·6	+24·6	+ 17·5

↑ Shear $= \frac{24\cdot6+17\cdot5}{12}$

$= 3\cdot5$ t

Design Example 10	8 of 12
Final Design of Column	

The stress condition in the upper segment AB and the lower segment BC is checked and found to be just satisfactory.

The design of the connection to transfer the vertical load from AB to BC and to simultaneously take care of the bending moment at the juncture point is now investigated. As a starting point, the vertical load of 41·5 t is transferred without consideration of bending moment with the addition of the ISLB 300, 37·7 kg to act as a diaphragm and to provide a reaction to the column section directly under the crane runway girder. Horizontal diaphragms are introduced at positions marked (4) in the figure and the moment capacity of these is checked. Since the diaphragms are more or less flexible in the vertical direction, these rivets are assumed to carry only a horizontal component of load. The moment capacity of these diaphragms is insufficient and additional rivets are added along line B-B to provide extra moment capacity as calculated in Sheet 9. The rivets along plane B-B are assumed to be good for vertical component of stress only. Since the moment arm of the rivets in the horizontal plane and those in the vertical planes are about equal they are assumed to share equally per rivet in the load.

Rechecked Combined Stress

Upper segment *AB*—ISWB 500, 95·2 kg

$$f_b = \frac{20{\cdot}6 \times 10^5}{2\ 091{\cdot}6} = 985 \text{ kg/cm}^2$$

$$\frac{{}^*331}{958} + \frac{985}{1\ 575} = 0{\cdot}972 < 1 \ \ldots\ldots \text{ OK.}$$

Lower segment *BC*—2 ISLB 450, 65·3 kg (*see* Sheet 5)

$$f_b = \frac{24{\cdot}6 \times 103 \times 48{\cdot}93}{273\ 700} = 453 \text{ kg/cm}^2$$

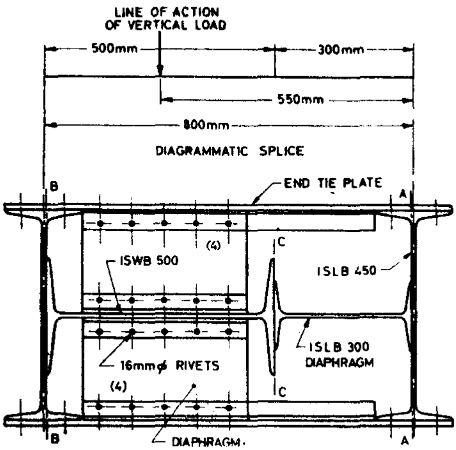
$$\frac{{}^\dagger 775}{1\ 156} + \frac{453}{1\ 575} = 0{\cdot}961 < 1 \ \ldots\ldots \text{ OK.}$$

Use 2—*ISLB* 450, 65·3 kg.

Compression splice AB to BC—First consider transfer of vertical load only.

Load on *AB* = 40 + 0·96 (wall) + 0·476 (self wt) = 41·5 t (*say*)

Reaction on the two ISLB 450, 65·3 kg sections (on lines *B-B* and *A-A*) would be half the total vertical load if the column *AB* were symmetrical in plane with respect to the column *BC*.



This being not the case (*see* the diagrammatical splice)

Taking moments on line *A-A*:

Reaction at *B-B* $= \frac{550}{800} \times 41{\cdot}5 = 28{\cdot}5$ t

Reaction at *A-A* $= 41{\cdot}5 - 28{\cdot}5 = 13$ t

The shear at *C-C* for which the joint between flange of ISLB 300, 37·7 kg and ISWB 500, 95·2 kg is subjected to is also = 13 t

Try 20 mm rivet on web of ISLB 450, 65·3 kg

Value in bearing $= 2{\cdot}1 \times 2{\cdot}360 \times 0{\cdot}86 = 4{\cdot}27$ t

Value in single shear $= \frac{\pi \times 2{\cdot}1^2}{4} \times 1{\cdot}025 = 3{\cdot}55$ t

∴ Value of single shear controls at *B-B*:

No. of rivets required $= \frac{28{\cdot}5}{3{\cdot}55} = 8$

Use ten 20-mm rivets at *B-B* connecting flange of ISWB 500, 95·2 kg to web of ISLB 450, 65·3 kg.

* *See* Sheet 4. † *See* Sheet 5.

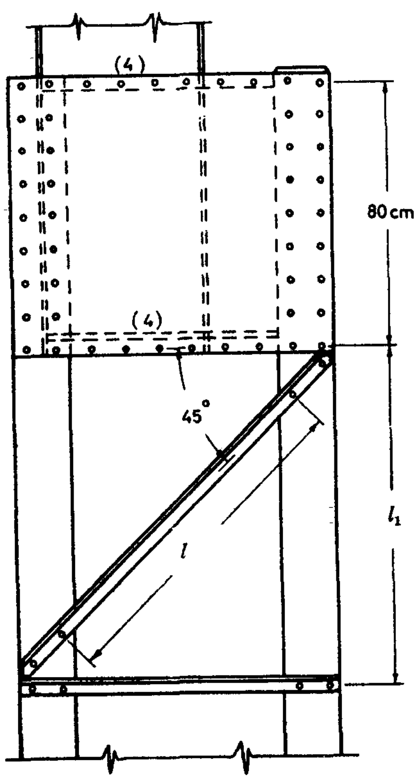
At A-A and C-C

No. of rivets required $= \dfrac{13}{3{\cdot}55} = 3{\cdot}67$

Use six 20-mm rivets at *A-A* and *C-C*, connecting flange of ISLB 300, 37·7 kg to web of ISLB 450, 65·3 kg and the other flange of ISLB 300, 37·7 kg to the flange of ISWB 500, 95·2 kg respectively.

Transfer of Bending Moment

Although rivets considered in the last sheet at *A-A* and *B-B* provide some moment resistance, check moment capacity at diaphragms 4-4 only.



Value in single shear of 5-16 mm rivets on each side $= \dfrac{\pi \times 1{\cdot}7^2}{4} \times 1{\cdot}025$

$= 2{\cdot}32$ t

10 rivets carry $10 \times 2{\cdot}32 = 23{\cdot}2$ t;

Lever arm $= 80$ cm

Rivets good for horizontal stress only.

Moment capacity of diaphragms (4) (through the ten rivets)

$= 23{\cdot}2 \times 80$
$= 1\,856$ cm· t

Moment to be resisted $= 2\,460$ cm·t (*see* Sheet 7)

Balance $= 604$ cm· t

NOTE — Maximum moment adds to stress in line *B-B*.

Increase the number of rivets of 20 mm diameter connecting flange of ISWB 500, 95·2 kg and web of ISLB 450, 65·3 kg in the vertical plane to 11.

$11 - 8 = 3$ rivets good for vertical stress only

Lever arm same as for diaphragms $= 80$ cm

Moment $= 3 \times 80 \times 3{\cdot}55$
$= 854 > 604$ cm· t
.....OK.

The lacing bars for the layout shown on Sheet 9 are checked both as compressive struts and tension members. The difference between this and the previous lacing design example under centric load (Design Example 7) is the additional shear induced by the lateral load and eccentricity of vertical load that is added to the 2·5 percent of axial load.

Design Example 10

Design of Lacing in Bottom Segment

Design of Lacing

Try 45° layout as shown in Sheet 9

Check local $\frac{l}{r_{1-1}}$; r_y of ISLB 450, 65·3 kg $= 3{\cdot}2$ cm $= r_{1-1}$

$l = 110$ cm

$\frac{l_1}{r_{1-1}} = \frac{110}{3{\cdot}2} = 35 < 0{\cdot}7 \times 56$ (of main member) .....OK.

$35 < 50$ .....OK (*see* **21.6** of IS : 800-1956)

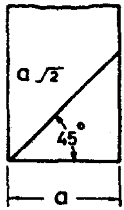
Shear

Load due to applied moment $= 3{\cdot}5$ t (*see* Sheet 7)

2·5 percent of axial load $= 0{\cdot}025 \times 125 = 3{\cdot}12$ t (*see* **21.2.1** of IS : 800-1956)

Total $= 6{\cdot}62$ t

Force in the lacing $= 6{\cdot}62 \times \sqrt{2} = 9{\cdot}36$ t



Try ISA 100 75, 6·0 mm with two rivets at each end.

$A = 10{\cdot}15$ cm²

$r_{min} = 1{\cdot}59$ cm

Effective length $= 80 \times \sqrt{2} = 113$ cm (*see* **21.2.3** of IS : 800-1956)

$l/r = \frac{113}{1{\cdot}59} = 71 < 145$.....OK (*see* **21.2.3** of IS : 800-1956)

$F_c = 1\ 090$ kg/cm² (Table I of IS : 800-1956)

Capacity of 2 angles $= 2 \times 1{\cdot}090 \times 10{\cdot}14 = 22{\cdot}1$ t $> 9{\cdot}36$ ..... OK, but over design

Try ISA 70 45, 5·0 mm

$A = 5{\cdot}52$ cm²

$r_{min} = 0{\cdot}96$ cm

$l/r = \frac{113}{0{\cdot}96} = 118 < 145$ ..... OK.

Capacity of 2 angle sections $= 2 \times 0{\cdot}726 \times 5{\cdot}52 = 8{\cdot}05$ t $< 9{\cdot}64$ t—No Good.

Use ISA 70 45, 6·0 mm

$A = 6{\cdot}56$ cm², Approx capacity as before $= 0{\cdot}726 \times 2 \times 6{\cdot}56 = 9{\cdot}5$ t.....OK.

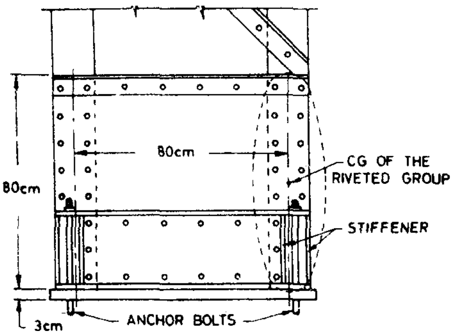
$r_{min} = 0{\cdot}96$ cm

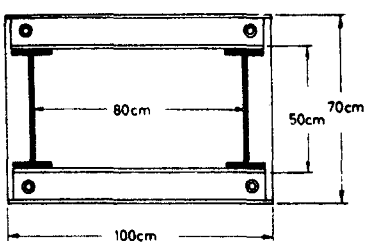
Use four 20-mm rivets value $= 4 \times 3{\cdot}55^{*} = 14{\cdot}2$ t $> 9{\cdot}36$ t .....OK.

* *See* Sheet 8.

Design Example 10	11 of 12
Design of Tie Plate	

IS : 800-1956 calls for end tie plates on compression members equal in length to the lateral breadth c/c of rivet groups attaching the tie to the main components. The layout shown at the centre of the sheet indicates the minimum length of the tie plates and may be made larger depending on how the lacing spacing works out in the final details. Four 25-mm diameter anchor bolts are shown and they engage a channel that is riveted to the end tie plates. It is well to have some excess of riveting in a detail of this kind so as to tie the end of the column into a single unit. The tie plate is first checked for its adequacy in transmitting the shear since it functions to take the place of a lacing bar in the end segment. The rivet group is found to be more than adequate. The anchorage bars are assumed to be pretensioned to their full permissible stress of 1 260 kg/cm² which is desirable to ensure adequate rotational rigidity. In order to check the moment capacity, it is assumed that a rectangular stress block is developed similar to what would be expected at ultimate load but here shown at the allowable working bearing pressure on a concrete pier of 55 kg/cm². Taking moments about the centre of the bearing plate, it is found that the moment capacity is more than double the actual applied moment. (It is obvious that the more conventional assumption of triangular block of pressure would also provide satisfactory resistance.) The additional safety with respect to moment is desirable and should provide adequate end fixity in accordance with design assumptions. The details for checking the thickness of bearing plate shown as 3 cm are also given. There is approximately a 10-cm overhang beyond the web of the main wide flange column members and this plate will distribute the load at less than the permissible 1 890 kg/cm² stress for bending in the bearing plate.





Tie Plate

Shear per tie plate $= \dfrac{{}^{*}6{\cdot}62\ \text{t}}{2}$

Shear per group of rivets $= \dfrac{6{\cdot}62}{4} = 1{\cdot}655$ t

Moment $= 1{\cdot}655 \times 80 = 132{\cdot}4$ cm·t

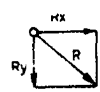
Try sixteen 20-mm rivets:

Average vertical spacing $= 9$ cm

Horizontal spacing $= 10$ cm

$$d^2 = x^2 + y^2$$

16 @ 5^2	=	400
4 @ $4{\cdot}5^2$	=	81
4 @ $13{\cdot}5^2$	=	729
4 @ $22{\cdot}5^2$	=	2 025
4 @ $31{\cdot}5^2$	=	3 969
		7 204



Max $R_x = \dfrac{132{\cdot}4}{7\ 204} \times 31{\cdot}5$

$= 0{\cdot}58\ \text{t} < 3{\cdot}55\ \text{t}$ shear .....OK.

No need to compute R_y. Rivets understressed in shear but needed to transfer load to base plate.

* *See* Sheet 10.

Design of Bearing Plate

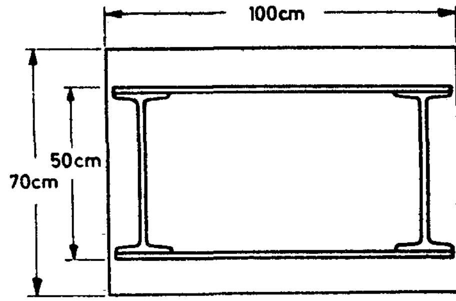
Assuming uniform load distribution:

Try 100 × 70 cm bearing plate.

Bearing pressure on concrete = 55 kg/cm²

$$t = \sqrt{\frac{3W}{F_b}\left(A^2 - \frac{B^2}{4}\right)}$$

(*see* **18.8.2** of IS : 800-1956)



$$t = \sqrt{\frac{3 \times 55}{1\ 890}(10^2 - 0^*)} = 3 \text{ cm}$$

Check Anchorage

Try 25-mm anchor bolts.

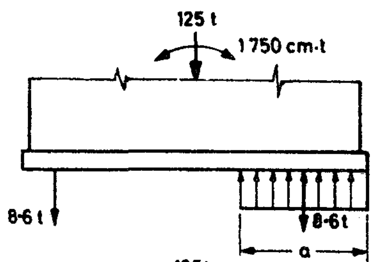
$$\text{Net area} = 0{\cdot}7 \times \frac{\pi \times 2{\cdot}5^2}{4}$$

$= 3{\cdot}43$ cm² (assuming net area = 0·7 gross area)

Assuming bearing on concrete base as 55 kg/cm² on rectangular stress block of width, *say*, a:

$$2 \times 8{\cdot}6\dagger + 125 = \frac{70 \times 55 \times a}{1\ 000}$$

$a = 37$ cm



Applied moment = 17·5 m·t (*see* Sheet 7)

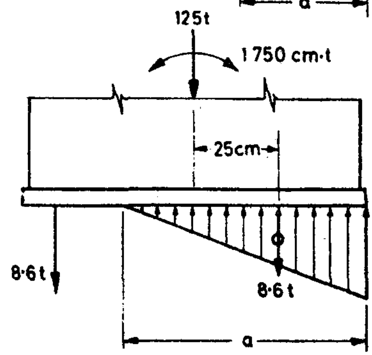
$$\text{Moment capacity} = 142{\cdot}6\,\frac{(100-37)}{2}$$

$= 4\ 500$ cm·t $> 1\ 750$ .....OK.

With the conventional triangular distribution

$$2 \times 8{\cdot}6 + 125 = \frac{70 \times 55 \times a}{2 \times 1\ 000}$$

$a = 74$ cm



Moment capacity = 142·6 (25)

$= 3\ 565$ cm·t $> 1\ 750$ cm·t .....OK.

* It is conservative to assume $B = 0$ (*see* sketch).

† Assume anchor bolts pretensioned to 1 260 kg/cm²: $2 \times 1{\cdot}260 \times 3{\cdot}43 = 8{\cdot}6$ t

SECTION V

CONCLUDING REMARKS CONCERNING COLUMN DESIGN

12. EFFICIENCY OF COMPRESSION MEMBERS

12.1 The design examples presented in this chapter have shown that for heavy loads and/or short lengths the centrally loaded column provides an effective stress carrying member. Because of the lesser stress that is permitted, the column is usually not quite as efficient as the tension member, except in cases where large deductions must be made for net section of rivet or bolt holes.

12.2 When small loads are to be carried over long distances, such as is the case in secondary bracing, the column becomes an inefficient member because of the very low stress that is permitted. When the permitted column stress for the minimum practicable l/r falls below 600 kg/cm^2, it is probable that the use of cross bracing, designed to carry the load in tension only, may be more economical than the use of a single diagonal that shall carry the load either in tension or compression. Thus column action is eliminated. There are many illustrations to be found in actual structures of such use of cross bracing. One such example is shown in Fig. 2 where light cross bracing is used for end wind load and crane braking, both in the plane of the roof and plane of the walls.

Figure 2 also shows crane runway girders carried by welded brackets attached to tapered columns as an alternate to stepped columns used in the previous Design Example 10. The use of such brackets may introduce more of a fatigue problem and will also cause greater eccentric moment than the use of the stepped columns.

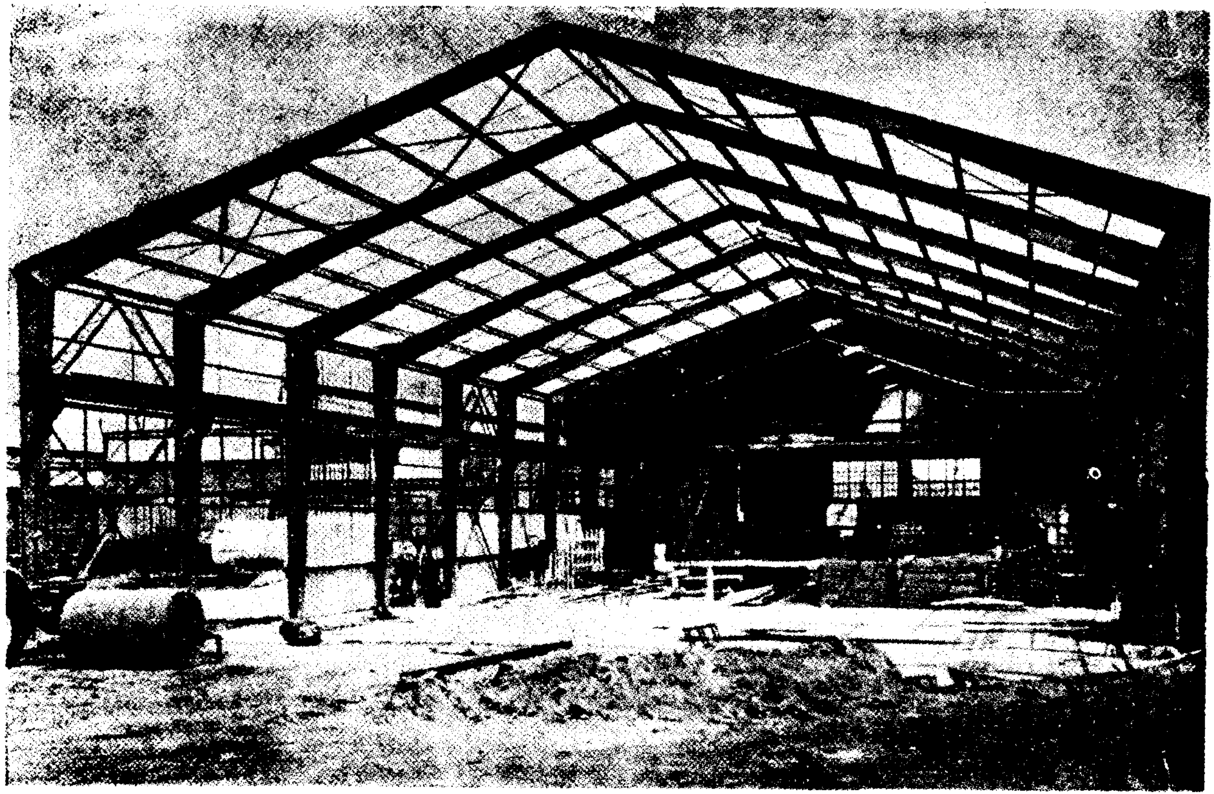


FIG. 2 COLUMNS WITH CRANE GANTRY

TABLE I ALLOWABLE AVERAGE STRESSES FOR AXIAL COMPRESSION

(*Clause* 2.2)

l/r	F_c		l/r	F_c	
	kg/cm²	tons/in.²		kg/cm²	tons/in.²
(1)	(2)	(3)	(1)	(2)	(3)
1	1 233	7·83	45	1 187	7·54
2	1 233	7·83	46	1 184	7·52
3	1 233	7·83	47	1 183	7·51
4	1 232	7·82	48	1 180	7·49
5	1 232	7·82	49	1 178	7·48
6	1 232	7·82	50	1 175	7·46
7	1 232	7·82	51	1 172	7·44
8	1 232	7·82	52	1 169	7·42
9	1 232	7·82	53	1 165	7·40
10	1 232	7·82	54	1 162	7·38
11	1 230	7·81	55	1 159	7·36
12	1 230	7·81	56	1 156	7·34
13	1 230	7·81	57	1 153	7·32
14	1 228	7·80	58	1 150	7·30
15	1 228	7·80	59	1 145	7·27
16	1 228	7·80	60	1 140	7·24
17	1 227	7·79	61	1 137	7·22
18	1 227	7·79	62	1 134	7·20
19	1 225	7·78	63	1 129	7·17
20	1 225	7·78	64	1 124	7·14
21	1 224	7·77	65	1 120	7·11
22	1 224	7·77	66	1 115	7·08
23	1 222	7·76	67	1 110	7·05
24	1 221	7·75	68	1 106	7·02
25	1 221	7·75	69	1 101	6·99
26	1 219	7·74	70	1 096	6·96
27	1 217	7·73	71	1 090	6·92
28	1 217	7·73	72	1 085	6·89
29	1 216	7·72	73	1 079	6·85
30	1 214	7·71	74	1 072	6·81
31	1 213	7·70	75	1 068	6·78
32	1 211	7·69	76	1 061	6·74
33	1 210	7·68	77	1 055	6·70
34	1 208	7·67	78	1 050	6·67
35	1 206	7·66	79	1 044	6·63
36	1 205	7·65	80	1 038	6·59
37	1 203	7·64	81	1 032	6·55
38	1 202	7·63	82	1 025	6·51
39	1 200	7·62	83	1 017	6·46
40	1 198	7·61	84	1 009	6·41
41	1 195	7·59	85	1 003	6·37
42	1 194	7·58	86	996	6·32
43	1 192	7·57	87	989	6·28
44	1 189	7·55	88	981	6·23

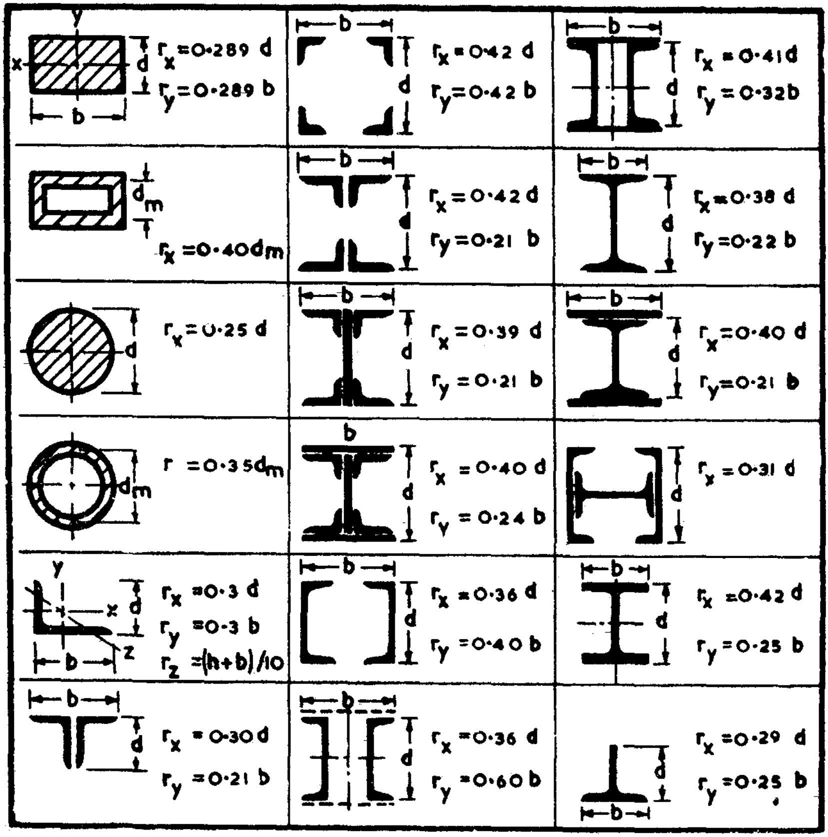
(*Continued*)

TABLE I ALLOWABLE AVERAGE STRESSES FOR AXIAL COMPRESSION—*Contd*

l/r	F_a		l/r	F_a	
	kg/cm²	tons/in.²		kg/cm²	tons/in.²
(1)	(2)	(3)	(1)	(2)	(3)
89	973	6·18	135	592	3·76
90	965	6·13	136	586	3·72
91	958	6·08	137	578	3·67
92	950	6·03	138	572	3 63
93	942	5·98	139	565	3·59
94	934	5·93	140	559	3·55
95	926	5·88	141	553	3·51
96	917	5·82	142	546	3·47
97	909	5·77	143	540	3·43
98	899	5·71	144	534	3·39
99	891	5·66	145	528	3·35
100	884	5·61	146	521	3·31
101	874	5·55	147	515	3·27
102	865	5·49	148	509	3·23
103	855	5·43	149	504	3·20
104	847	5·38	150	499	3·17
105	838	5·32	151	491	3·12
106	830	5·27	152	485	3·08
107	821	5·21	153	479	3·04
108	813	5·16	154	472	3·00
109	803	5·10	155	466	2·96
110	795	5·05	156	461	2·93
111	786	4·99	157	455	2·89
112	776	4·93	158	449	2·85
113	769	4·88	159	444	2·82
114	759	4·82	160	438	2·78
115	751	4·77	161	433	2·75
116	742	4·71	162	427	2·71
117	734	4·66	163	422	2·68
118	726	4·61	164	416	2·64
119	717	4·55	165	411	2·61
120	709	4·50	166	406	2·58
121	701	4·45	167	402	2·55
122	693	4·40	168	397	2·52
123	685	4·35	169	392	2·49
124	676	4·29	170	387	2·46
125	668	4·24	171	381	2·42
126	660	4·19	172	376	2·39
127	652	4·14	173	372	2·36
128	644	4·09	174	367	2·33
129	636	4·04	175	362	2·30
130	630	4·00	176	357	2·27
131	622	3·95	177	353	2·24
132	614	3·90	178	348	2·21
133	608	3·86	179	345	2·19
134	600	3·81	180	340	2·16

TABLE II APPROXIMATE RADII OF GYRATION

(*Clauses* 2.3 *and* 2.4)



APPENDIX A

(*See* Foreword)

INDIAN STANDARDS ON PRODUCTION, DESIGN AND USE OF STEEL IN STRUCTURES

ISI has so far issued the following Indian Standards in the field of production, design and utilization of steel and welding:

IS : 800-1956 CODE OF PRACTICE FOR USE OF STRUCTURAL STEEL IN GENERAL BUILDING CONSTRUCTION

IS : 801-1958 CODE OF PRACTICE FOR USE OF COLD FORMED LIGHT GAUGE STEEL STRUCTURAL MEMBERS IN GENERAL BUILDING CONSTRUCTION

IS : 804-1958 SPECIFICATION FOR RECTANGULAR PRESSED STEEL TANKS

IS : 806-1957 CODE OF PRACTICE FOR USE OF STEEL TUBES IN GENERAL BUILDING CONSTRUCTION

IS : 808-1957 SPECIFICATION FOR ROLLED STEEL BEAM, CHANNEL AND ANGLE SECTIONS

IS : 812-1957 GLOSSARY OF TERMS RELATING TO WELDING AND CUTTING OF METALS

IS : 813-1961 SCHEME OF SYMBOLS FOR WELDING (*Amended*)

IS : 814-1957 SPECIFICATION FOR COVERED ELECTRODES FOR METAL ARC WELDING OF MILD STEEL

IS : 815-1956 CLASSIFICATION AND CODING OF COVERED ELECTRODES FOR METAL ARC WELDING OF MILD STEEL AND LOW ALLOY HIGH-TENSILE STEELS

IS : 816-1956 CODE OF PRACTICE FOR USE OF METAL ARC WELDING FOR GENERAL CONSTRUCTION IN MILD STEEL

IS : 817-1957 CODE OF PRACTICE FOR TRAINING AND TESTING OF METAL ARC WELDERS

IS : 818-1957 CODE OF PRACTICE FOR SAFETY AND HEALTH REQUIREMENTS IN ELECTRIC AND GAS WELDING AND CUTTING OPERATIONS

IS : 819-1957 CODE OF PRACTICE FOR RESISTANCE SPOT WELDING FOR LIGHT ASSEMBLIES IN MILD STEEL

IS : 1173-1957 SPECIFICATION FOR ROLLED STEEL SECTIONS, TEE BARS

IS : 1179-1957 SPECIFICATION FOR EQUIPMENT FOR EYE AND FACE PROTECTION DURING WELDING

IS : 818-1968 CODE OF PRACTICE FOR SAFETY AND HEALTH REQUIREMENTS IN ELECTRIC AND GAS WELDING AND CUTTING OPERATIONS (*First revision*)

IS : 819-1957 CODE OF PRACTICE FOR RESISTANCE SPOT WELDING FOR LIGHT ASSEMBLIES IN MILD STEEL

IS : 1173-1967 SPECIFICATION FOR HOT ROLLED AND SILT STEEL, TEE BARS (*First revision*)

IS : 1179-1967 SPECIFICATION FOR EQUIPMENT FOR EYE AND FACE PROTECTION DURING WELDING (*First revision*)

IS : 1181-1967 QUALIFYING TESTS FOR METAL ARC WELDERS (ENGAGED IN WELDING STRUCTURES OTHER THAN PIPES) (*First revision*)

IS : 1182-1967 RECOMMENDED PRACTICE FOR RADIOGRAPHIC EXAMINATION OF FUSION WELDED BUTT JOINTS IN STEEL PLATES (*First revision*)

IS : 1252-1958 SPECIFICATION FOR ROLLED STEEL SECTIONS, BULB ANGLES

IS : 1261-1959 CODE OF PRACTICE FOR SEAM WELDING IN MILD STEEL

IS : 1278-1972 SPECIFICATION FOR FILLER RODS AND WIRES FOR GAS WELDING (*Second revision*)

IS : 1323-1966 CODE OF PRACTICE FOR OXY-ACETYLENE WELDING FOR STRUCTURAL WORK IN MILD STEEL (*Revised*)

IS : 1395-1971 SPECIFICATION FOR MOLYBDENUM AND CHROMIUM-MOLYBDENUM-VANADIUM LOW ALLOY STEEL ELECTRODES FOR METAL ARC WELDING (*Second revision*)

IS : 1442-1964 SPECIFICATION FOR COVERED ELECTRODES FOR THE METAL ARC WELDING OF HIGH TENSILE STRUCTURAL STEEL (*Revised*)

APPENDIX B

(*See* Foreword)

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