10 DESIGN OF AXIALLY LOADED COLUMNS

1.0 INTRODUCTION

In an earlier chapter, the behaviour of practical columns subjected to axial compressive loading was discussed and the following conclusions were drawn.

- Very short columns subjected to axial compression fail by yielding. Very long columns fail by buckling in the Euler mode.
- Practical columns generally fail by inelastic buckling and do not conform to the assumptions made in Euler theory. They do not normally remain linearly elastic upto failure unless they are very slender
- Slenderness ratio (λ/r) and material yield stress (f_y) are dominant factors affecting the ultimate strengths of axially loaded columns.
- The compressive strengths of practical columns are significantly affected by (i) the initial imperfection (ii) eccentricity of loading (iii) residual stresses and (iv) lack of defined yield point and strain hardening. Ultimate load tests on practical columns reveal a scatter band of results shown in Fig. 1. A lower bound curve of the type shown therein can be employed for design purposes.

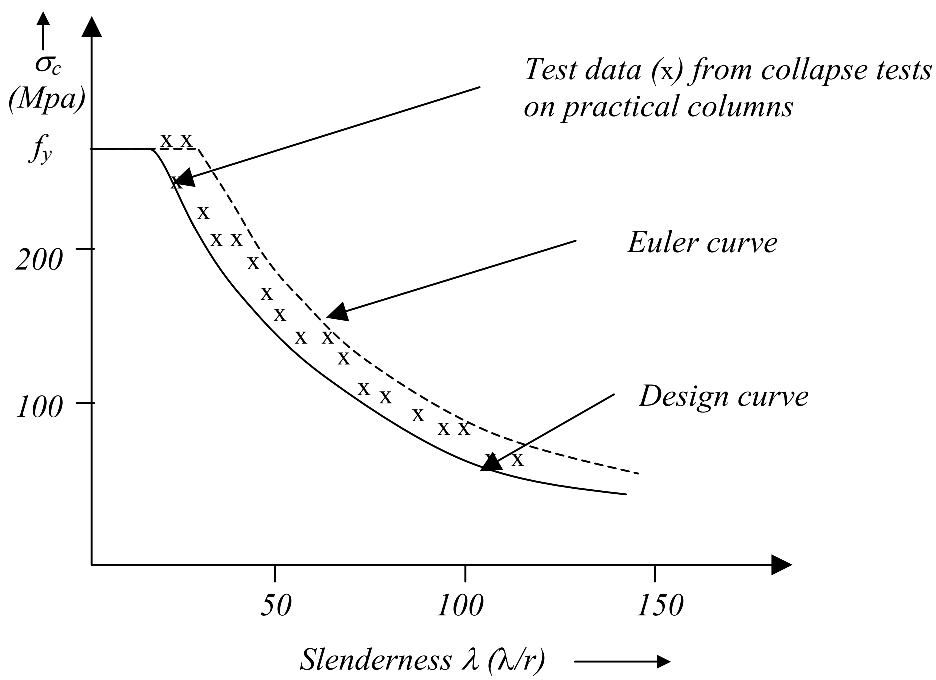


Fig. 1 Typical column design curve

2.0 HISTORICAL REVIEW

Based on the studies of Ayrton & Perry (*1886*), the British Codes had traditionally based the column strength curve on the following equation.

$$(f_y - \sigma_c)\ (\sigma_e - \sigma_c) = \eta \cdot \sigma_e \cdot \sigma_c \tag{1}$$

where

f_y = yield stress

σ_c = compressive strength of the column obtained from the positive root of the above equation

σ_e = Euler buckling stress given by $\dfrac{\pi^2 E}{\lambda^2}$ (1a)

η = a parameter allowing for the effect of lack of straightness and eccentricity of loading.

λ = Slenderness ratio given by (λ/r)

In the deviation of the above formula, the imperfection factor η was based on

$$\eta = \frac{y \cdot \Delta}{r^2} \tag{2}$$

where y = the distance of centroid of the cross section to the extreme fibre of the section.

Δ = initial bow or lack of straightness

r = radius of gyration.

Based on about *200* column tests, Robertson (*1925*) concluded that the initial bow (Δ) could be taken as *length of the column/1000* consequently η is given by

$$\eta = \left(0.001\,\frac{y}{r}\right) . \left(\frac{\lambda}{r}\right)$$

$$= \alpha \left(\frac{\lambda}{r}\right) = \alpha \cdot \lambda \tag{3}$$

where α is a parameter dependent on the shape of the cross section.

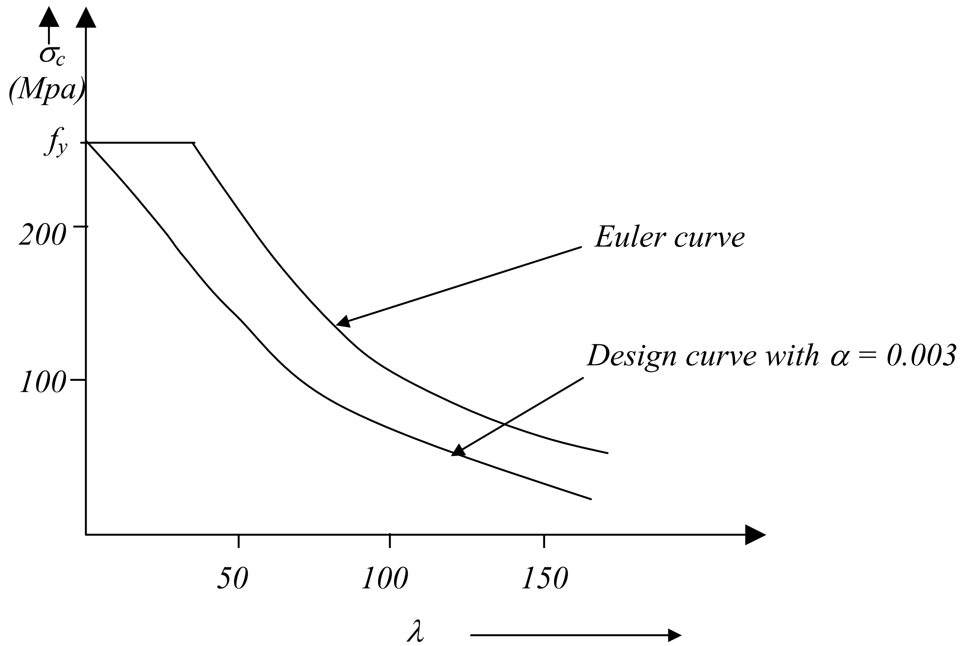


Fig.2 Robertson's Design Curve

Robertson evaluated the mean values of α for many sections as given in Table 1:

Table1: α values Calculated by Robertson

Column type	***α Values***
Beams & Columns about the major axis	0.0012
Rectangular Hollow sections	0.0013
Beams & Universal columns about the minor axis	0.0020
Tees in the plane of the stem	0.0028

He concluded that the lower bound value of $\alpha = 0.003$ was appropriate for column designs. This served as the basis for column designs in Great Britain until recently. The design curve using this approach is shown in Fig. 2. The Design method is termed "Perry-Robertson approach".

3.0 MODIFICATION TO THE PERRY ROBERTSON APPROACH

3.1 Stocky Columns

It has been shown previously that very stocky columns (e.g. stub columns) resisted loads in excess of their squash load of $f_y.A$ (i.e. theoretical yield stress multiplied by the area of the column). This is because the effect of strain hardening is predominant in low values of slenderness *(λ)*. Equation (1) will result in column strength values lower than f_y even in very low slenderness cases. To allow empirically for this discrepancy, recent British and European Codes have made the following modification to equation 3 given by

$$\eta = \alpha \lambda$$

In the unmodified form this will cause a drop in the calculated value of column strength even for very low values of slenderness. Such columns actually fail by squashing and there is no drop in observed strengths in such very short columns. By modifying the slenderness, λ to $(\lambda - \lambda_0)$ we can introduce a plateau to the design curve at low slenderness values. In generating the British Design (BS: 5950 Part-1) curves

$\lambda_0 = 0.2 \left(\pi \sqrt{E / f_y}\right)$ was used as an appropriate fit to the observed test data, so that we

obtain the failure load (equal to squash load) for very low slenderness values. Thus in calculating the elastic critical stress, we modify the formula used previously as follows:

$$\sigma_e = \frac{\pi^2 E}{(\lambda - \lambda_0)^2} \text{ for all values of } \lambda > \lambda_0 \qquad (4)$$

Note that no calculations for σ_e is needed when $\lambda \leq \lambda_e$ as the column would fail by squashing at f_y.

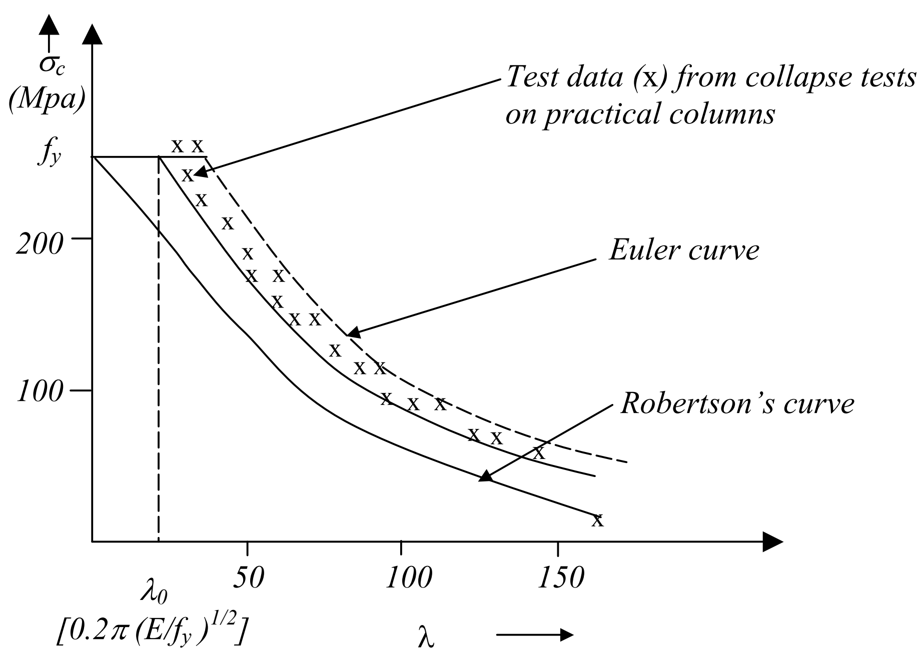


Fig.3 Strut curve with a plateau for low slenderness values

3.2 Influence of Residual Stresses

Reference was made earlier to the adverse effect of locked-in residual stresses on column strengths (see Fig. 4). Studies on columns of various types carried out by the European Community have resulted in the recommendation for adopting a family of design curves rather than a single "*Typical Design Curve*" shown in Fig. 3. Typically four column curves are suggested in British and European codes for the different types of sections commonly used as compression members [See Fig. 5(*a*)]. In these curves, $\eta = \alpha(\lambda - \lambda_0)$

where $\lambda_0 = 0.2\pi\sqrt{E/f_y}$ and the α values are varied corresponding to various sections.

Thus all column designs are to be carried out using the strut curves given in [Fig. 5(*a*)].

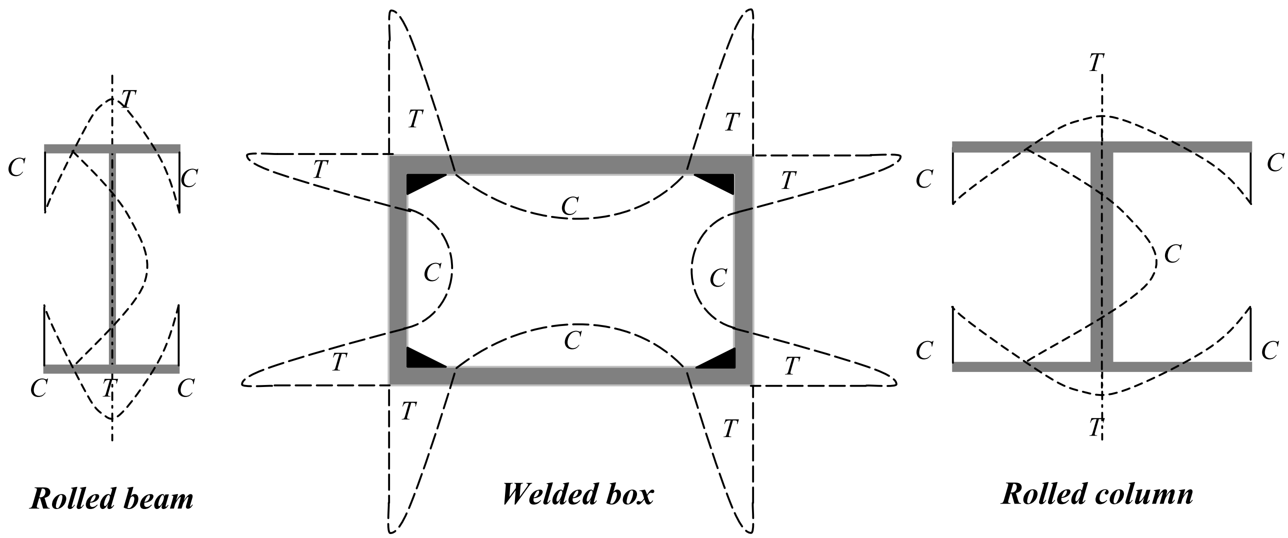


Fig. 4 Distribution of residual stresses

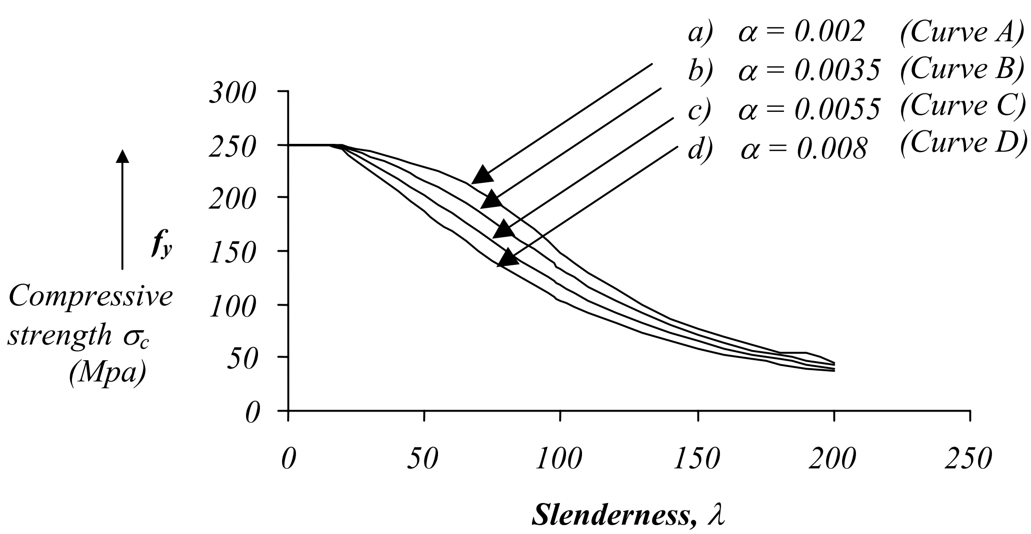


Fig. 5(a) Compressive strength curves for struts for different values of α

The selection of an appropriate curve is based on cross section and suggested curves are listed in Table 2.

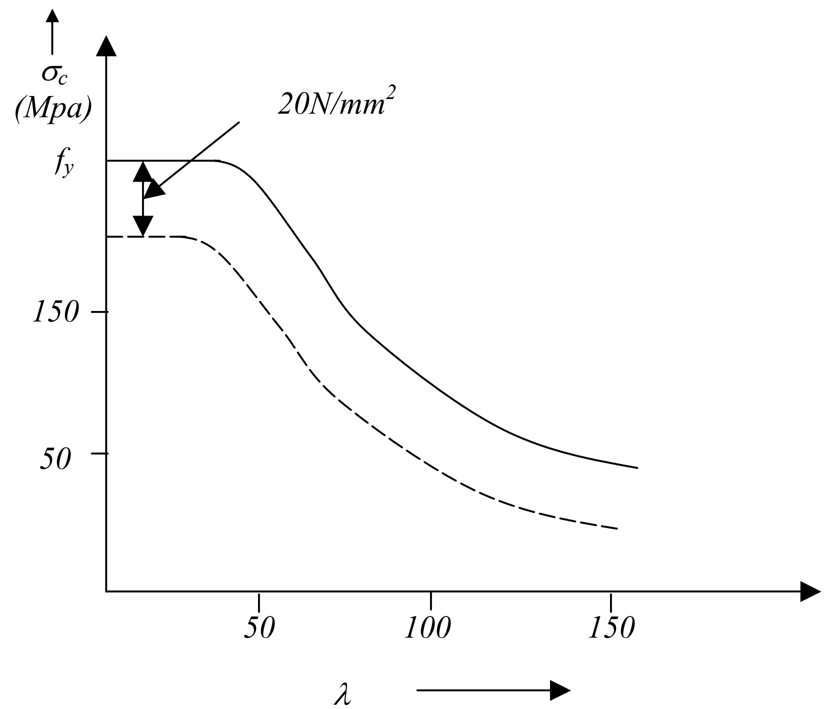


Fig. 5(b) Compressive strength of welded sections

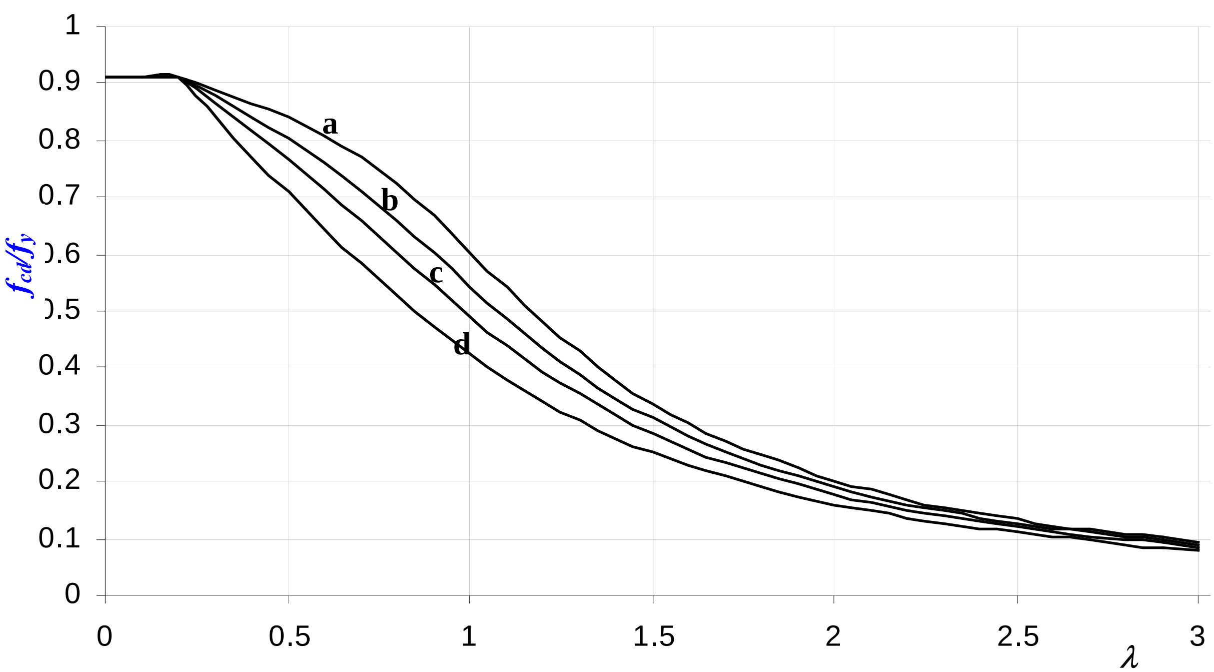


Fig. 5(c) Column Buckling Curves as per IS: 800

Table 2: Choice of appropriate values of α

Sections	Axis of buckling	
	X -X	***Y - Y***
Hot rolled structural hollow sections	α = 0.002 (Curve A)	α = 0.002 (Curve A)
Hot rolled I section	α = 0.002 (Curve A)	α = 0.0035 (Curve B)
Welded plate I section (up to 40 mm thick) I section (above 40 mm thick)	 α = 0.0035 (Curve B) α = 0.0035 (Curve B)	 α = 0.0055 (Curve C) α = 0.008 (Curve D)
Welded Box Section (Up to 40 mm thick) (Over 40 mm thick)	 α = 0.0035 (Curve B) α = 0.0055 (Curve C)	 α = 0.0035 (Curve B) α = 0.0055 (Curve C)
Rolled I section with Welded cover plates (Up to 40mm thick) (Over 40mm thick)	α = 0.0035 (Curve B) α = 0.0055 (Curve C)	α = 0.002 (Curve A) α = 0.0035(Curve B)
Rolled angle, Channel, T section Compound sections - Two rolled sections back to back, Battened or laced sections	Check buckling about ANY axis With α = 0.0055 (Curve C)	

Note: For sections fabricated by plates by welding the Value of f_y should be reduced by 20 N/mm^2 [See Fig. 5(*b*)].

For computational convenience, formulae linking σ_c and λ are required. The lower root of equation (1) [based on Perry - Robertson approach] represents the strut curves given Fig. 3 and BS: 5950 Part - 1.

$$\sigma_c = \phi - \sqrt{\phi^2 - f_y \sigma_e} \; \leq \; f_y \tag{5}$$

$$where, \; \phi = \frac{f_y + (\eta + 1)\sigma_e}{2} \tag{6a}$$

$$and \;\; \eta = (\lambda - \lambda_0)\alpha \tag{6b}$$

3.3 Types of Column Sections

Steel suppliers manufacture several types of sections, each type being most suitable for specific uses. Some of these are described below. It is important to note that columns may buckle about *Z*, *Y*, *V* or *U* axis. It is necessary to check the safety of the column about several axes, so that the lowest load that triggers the onset of collapse may be identified.

Universal Column *(UC)* sections have been designed to be most suitable for compression members. They have broad and relatively thick flanges, which avoid the problems of local buckling. The open shape is ideal for economic rolling and facilitates easy beam-to-column connections. The most optimum theoretical shape is in fact a ***circular hollow section (CHS),*** which has no weak bending axis. Although these have been employed in large offshore structures like oil platforms, their use is somewhat limited because of high connection costs and comparatively weaker in combined bending and axially compressive loads. ***Rectangular Hollow Sections (RHS)*** have been widely used in multi-storey buildings satisfactorily. For relatively light loads, (e.g. Roof trusses) ***angle sections*** are convenient as they can be connected through one leg. Columns, which are subjected to bending in addition to axial loads, are designed using ***Universal beams (UB)***.

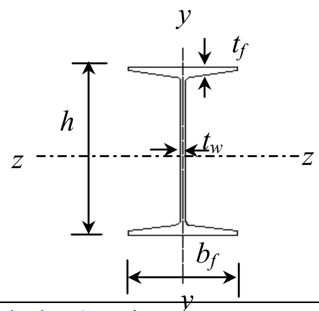
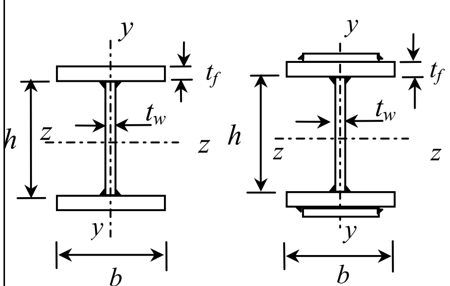
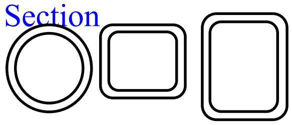
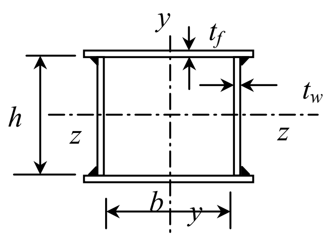
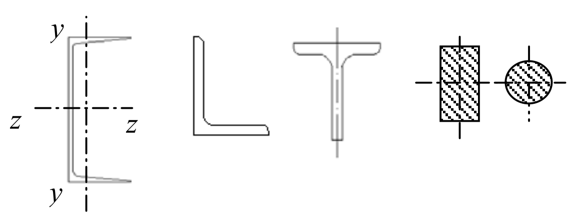
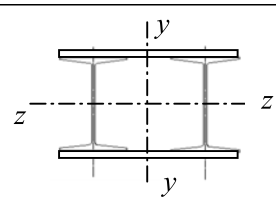
3.4 Heavily Welded Sections

Although both hot rolled sections and welded sections have lock-in residual stresses, the distribution and magnitude differ significantly. Residual stresses due to welding are very high and can be of greater consequence in reducing the ultimate capacity of compression members.

3.5 Stipulations of IS: 800

The stipulations of IS: 800 follow the same methodology as detailed above. For various types of column cross sections including Indian Standard rolled steel sections (as against Universal Column sections), CHS, SHS, RHS and Heavily Welded sections, IS: 800 recommends the classifications following the column buckling curves a, b, c and d as detailed in fig. 5(c) above. Apart from types of column cross-sections, the respective geometric dimensions of individual structural elements and their corresponding limits also guide Buckling Class. For example, if the ratio of overall height is to the overall width of the flange of rolled I section i.e. *h/b* is greater than 1.2 and the thickness of flange is less than 40 mm, the buckling class corresponding to axis z-z will be guided by Curve a of Fig. 5 (c). This shows for same slenderness ratio, more reserve strength is available for the case described above in comparison to built-up welded sections. For various types of column cross-sections, Table 3 (Table 7.2 of Revised IS: 800) defines the buckling classes with respect to their respective limits of height to width ratio, thickness of flange and the axis about which the buckling takes place (to be determined based on the column buckling curves as indicated in fig. 5 (c) corresponding to both major and minor axis):

TABLE 3: BUCKLING CLASS OF CROSS SECTIONS

Cross Section	Limits	Buckling about axis	Buckling Class
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	40 mm < $t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (Except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

4.0 EFFECTIVE LENGTH OF COLUMNS

The effective length, KL, is calculated from the actual length, L, of the member, considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from center to center of its intersections with the supporting members in the plane of the buckling deformation, or in the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end.

Effective Length – Where the boundary conditions in the plane of buckling can be assessed, the effective length, KL, can be calculated on the basis of Table 7.5. Where frame analysis does not consider the equilibrium of a framed structure in the deformed shape (Second-order analysis or Advanced analysis), the effective length of compression members in such cases can be calculated using the procedure given in Appendix E.1. The effective length of stepped column in individual buildings can be calculated using the procedure given in Appendix E.2.

Eccentric Beam Connection – In cases where the beam connections are eccentric in plan with respect to the axes of the column, the same conditions of restraint as in concentric connection, shall be deemed to apply, provided the connections are carried across the flange or web of the columns as the case may be, and the web of the beam lies within, or in direct contact with the column section. Where practical difficulties prevent this, the effective length shall be taken as equal to the distance between points of restraint, in non-sway frames.

Stipulations of IS: 800 – Method for determining Effective Length for Stepped Columns

Single Stepped Columns – Effective length in the plane of stepping (bending about axis z-z) for bottom and top parts for single stepped column shall be taken as given in Table E.2

Note: The provisions of E.2.1 are applicable to intermediate columns as well with stepping on either side, provided appropriate values of I1and I2 are taken

Double Stepped Columns – Effective lengths in the plane of stepping (bending about axis z-z) for bottom, middle and top parts for a double stepped column shall be taken as per the stipulations of Appendix E3

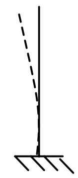
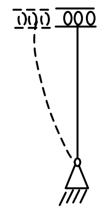
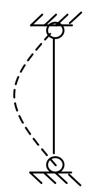
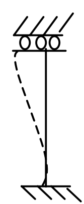
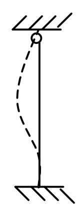
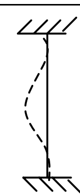
Coefficient K_1 for effective length of bottom part of double stepped column shall be taken from the formula:

$$K_1 = \sqrt{\frac{t_1 K_1^2 + \left(t_2 K_2^2 + K_3^2\right) \times \left(1 + n_2\right)^2 \times \dfrac{I_1}{I'_{av}}}{1 + t_1 + t_2}}$$

where

K_1, K_2, and K_3 are taken from Table E.6,

TABLE 4: EFFECTIVE LENGTH OF PRISMATIC COMPRESSION MEMBERS

Boundary Conditions				Schematic representation	Effective Length
At one end		At the other end			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Restrained	Free		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65 L

Note – L is the unsupported length of the compression member (7.2.1).

Designs of columns have to be checked using the appropriate effective length for buckling in both strong and weak axes. A worked example illustrating this concept is appended to this chapter.

5.0 STEPS IN DESIGN OF AXIALLY LOADED COLUMNS AS PER IS: 800

The procedure for the design of an axially compressed column as stipulated in IS: 800 is as follows:

(i) Assume a suitable trial section and classify the section in accordance with the classification as detailed in Table 3.1 (Limiting Width to Thickness Ratios) of the Chapter 3 of IS: 800. (If, the section is slender then apply appropriate correction factor).

(ii) Calculate effective sectional area, $\boldsymbol{A_e}$ as defined in Clause 7.3.2 of IS: 800

(iii) Calculate effective slenderness ration, KL/r, ratio of effective length KL, to appropriate radius of gyration, r

(iv) Calculate λ from the equation, λ = non-dimensional effective slenderness ratio = $\sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

(v) Calculate ϕ from the equation, $\phi = 0.5[1+\alpha(\lambda - 0.2)+ \lambda^2]$
Where,
α = Imperfection factors for various Column Buckling Curves a, b, c and d are given in the following Table: (Table 7.1 of IS: 800)

TABLE 5: IMPERFECTION FACTOR, α

Buckling Class	**a**	**b**	**c**	**d**
α	0.21	0.34	0.49	0.76

(vi) Calculate χ from equation, $\chi = \left[1 \Big/ \left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right] \right]$

(vii) Choose appropriate value of Partial safety factor for material strength, γ_{m0} from Table 5.2 of Chapter 5 of IS: 800

(viii) Calculate design stress in compression, $\boldsymbol{f_{cd}}$, as per the following equation (Clause **7.1.2.1** of IS: 800):

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} \qquad = \qquad \chi f_y / \gamma_{m0} \qquad \le f_y / \gamma_{m0}$$

(ix) Compute the load $\boldsymbol{P_d}$, that the compression member can resist $\boldsymbol{P_d = A_e f_{cd}}$

(x) Calculate the factored applied load and check whether the column is safe against the given loading. The most economical section can be arrived at by trial and error, i.e. repeating the above process.

6.0 CROSS SECTIONAL SHAPES FOR COMPRESSION MEMBERS AND BUILT- UP COLUMNS

Although theoretically we can employ any cross sectional shape to resist a compressive load we encounter practical limitations in our choice of sections as only a limited number of sections are rolled by steel makers and there are sometimes problems in connecting them to the other components of the structure. Another limitation is due to the adverse impact of increasing slenderness ratio on compressive strengths; this virtually excludes the use of wide plates, rods and bars, as they are far too slender. It must be specially noted that all values of slenderness ratio referred to herein are based on the least favourable value of radius of gyration, so that (λ/r) is the highest value about any axis.

6.1 Rolled Steel Sections

Some of the sections employed as compression members are shown in Fig. 6. Single angles [Fig 6(*a*)] are satisfactory for bracings and for light trusses. Top chord members of roof trusses are usually made up of twin angles back to back [Fig 6(*b*)]

Double angle sections shown in Fig. 6(*b*) are probably the most commonly used members in light trusses. The pair of angles used has to be connected together, so they will act as one unit. Welds may be used at intervals – with a spacer bar between the connecting legs. Alternately “ stitch bolts”, washers and “ring fills” are placed between the angles to keep them at the proper distance apart (e.g. to enable a gusset to be connected).

When welded roof trusses are required, there is no need for gusset plates and T sections [Fig 6(*c*)] can be employed as compression members.

Single channels or C-sections [Fig. 6(*d*)] are generally not satisfactory for use in compression, because of the low value of radius of gyration. They can be used if they could be supported in a suitable way in the weak direction.

Circular hollow sections [Fig. 6(*e*)] are perhaps the most efficient as they have equal values of radius of gyration about every axis. But connecting them is difficult but satisfactory methods have been evolved in recent years for their use in tall buildings.

The next best in terms of structural efficiency will be the square hollow sections (SHS) and rectangular hollow sections, [Fig. 6(*f*)] both of which are increasingly becoming popular in tall buildings, as they are easily fabricated and erected. Welded tubes of circular, rectangular or square sections are very satisfactory for use as columns in a long series of windows and as short columns in walkways and covered warehouses. For many structural applications the weight of hollow sections required would be only *50%* of that required for open profiles like I or C sections.

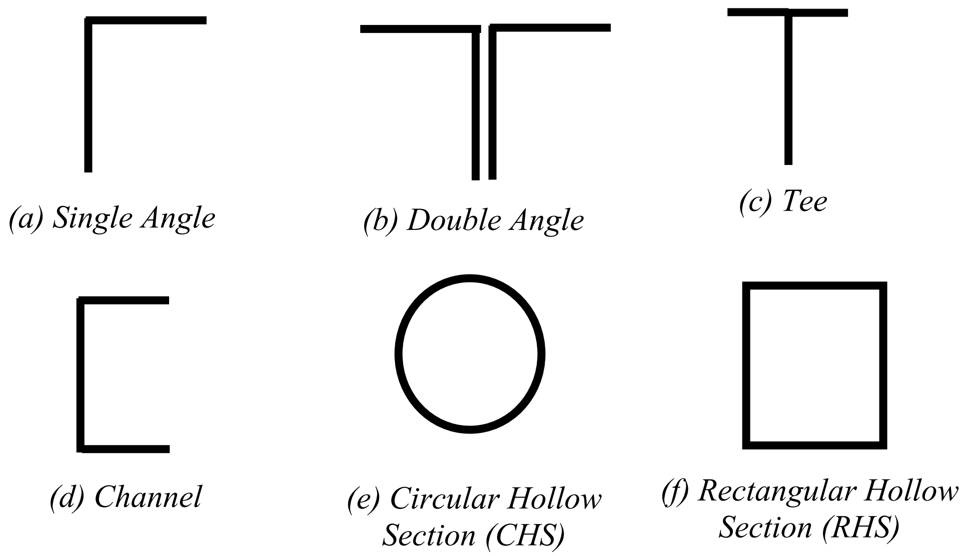


Fig 6: Cross Section Shapes for Rolled Steel Compression Members

The following general guidance is given regarding connection requirements:

When compression members consist of different components, which are in contact with each other and are bearing on base plates or milled surfaces, they should be connected at their ends with welds or bolts. When welds are used, the weld length must be not less than the maximum width of the member. If bolts are used they should be spaced longitudinally at less than *4* times the bolt diameter and the connection should extend to at least *1 ½* times the width of the member.

Single angle discontinuous struts connected by a single bolt are rarely employed. When such a strut is required, it may be designed for *1.25* times the factored axial load and the effective length taken as centre to centre of the intersection at each end. Single angle discontinuous struts connected by two or more bolts in line along the member at each end may be designed for the factored axial load, assuming the effective length to be *0.85* times the centre to centre distance of the intersection at each end.

For double angle discontinuous struts connected back to back to both sides of a gusset or section by not less than two bolts or by welding, the factored axial load is used in design, with an effective length conservatively chosen. (A value between *0.7* and *0.85* depending upon the degree of restraint provided at the ends).

All double angle struts must be tack bolted or welded. The spacing of connectors must be such that the largest slenderness ratio of each component member is neither greater than *60* nor less than *40*. A minimum of two bolts at each end and a minimum of two additional connectors spaced equidistant in between will be required. Solid washers or packing plates should be used in-between if the leg width of angles exceed *125 mm*.

For member thickness upto *10 mm*, *M16* bolts are used; otherwise *M20* bolts are used. Spacing of tack bolts or welds should be less than *600 mm*.

The following guide values are suggested for initial choice of members:

(i) Single angle size : *1/30* of the length of the strut $\left(\lambda/r \approx 150\right)$

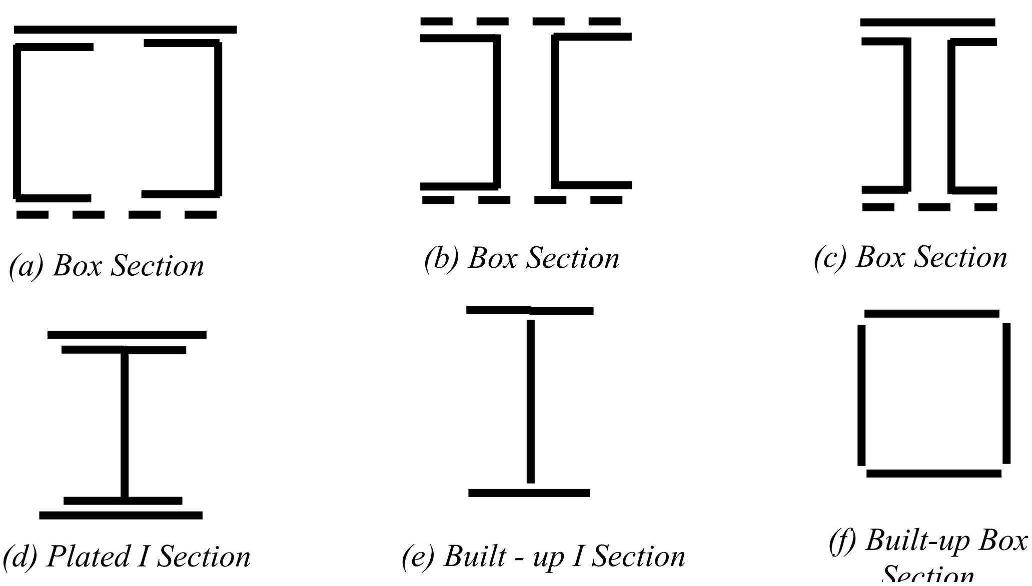
(ii) Double angle size : *1/35* of the length of strut $\left(\lambda/r \approx 100-120\right)$

(iii) Circular hollow sections diameter = *1/40* length $\left(\lambda/r \approx 100\right)$

6.2 Built-up or fabricated Compression Members

When compression members are required for large structures like bridges, it will be necessary to use built-up sections. They are particularly useful when loads are heavy and members are long (e.g. top chords of Bridge Trusses). Built up sections [illustrated in Fig. 7(*a*) and 7(*b*)] are popular in India when heavy loads are encountered. The cross section consists of two channel sections connected on their open sides with some type of lacing or latticing (dotted lines) to hold the parts together and ensure that they act together as one unit. The ends of these members are connected with "batten plates" which tie the ends together. Box sections of the type shown in Fig. 7(*a*) or 7(*b*) are sometimes connected by solid plates also (represented by straight lines).

A pair of channels connected by cover plates on one side and latticing on the other [Fig.7(c)] is sometimes used as top chords of bridge trusses. The gussets at joints can be conveniently connected to the inside of the channels. Plated I sections or built-up I sections are used when the available rolled I sections do not have sufficient strengths to resist column loads [Fig 7(*d*) and 7(*e*)]. For very heavy column loads, a welded built up section [See Fig. 7(*f*)] is quite satisfactory.



(a) Box Section *(b) Box Section* *(c) Box Section*

(d) Plated I Section *(e) Built - up I Section* *(f) Built-up Box Section*

Fig 7: Cross Section Shapes for Built - up or fabricated Compression Members

Built up columns made up of solid webs:

In these columns the webs are solid and continuous [See Fig 7(*d*), 7(*e*), and 7(*f*)]. Flange plates or channels may be used in combination with rolled sections to enhance the load resistance of the commonly available sections, which are directly welded or bolted to each other. For preliminary calculations, approximate values of radii of gyration given in Fig. 8 for various built-up sections may be employed.

The lateral dimension of the column is generally chosen at around *1/10* to *1/15* of the height of the column. For purposes of detailing the connection between the flange cover plates or the outer rolled sections to the flanges of the main rolled section, it is customary to design the fasteners for a transverse shear force equal to *2.5%* of the compressive load of the column. (Connection Design is dealt later in this resource).

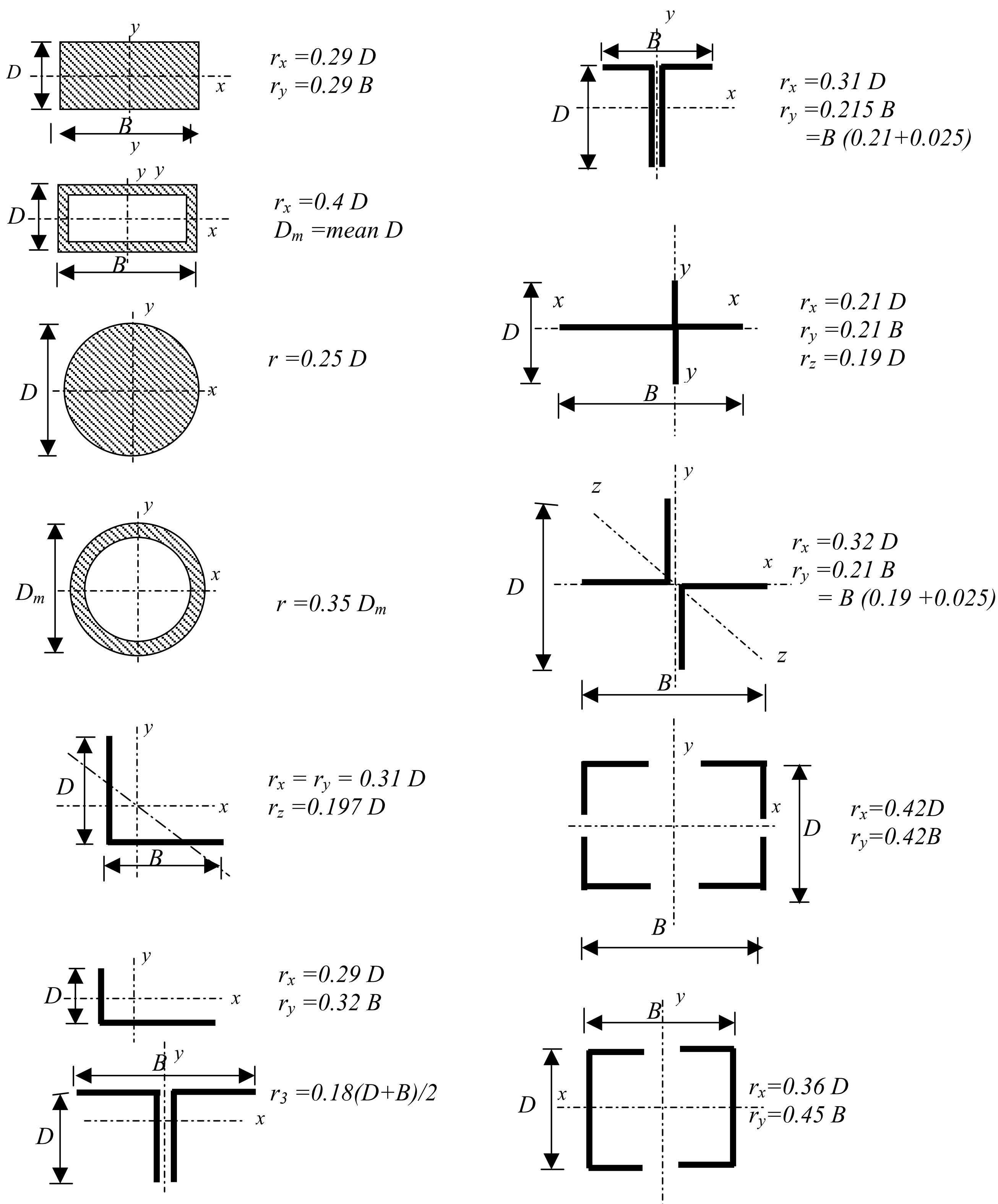


Fig 8: Approximate radii of gyration

(Continued in next page)

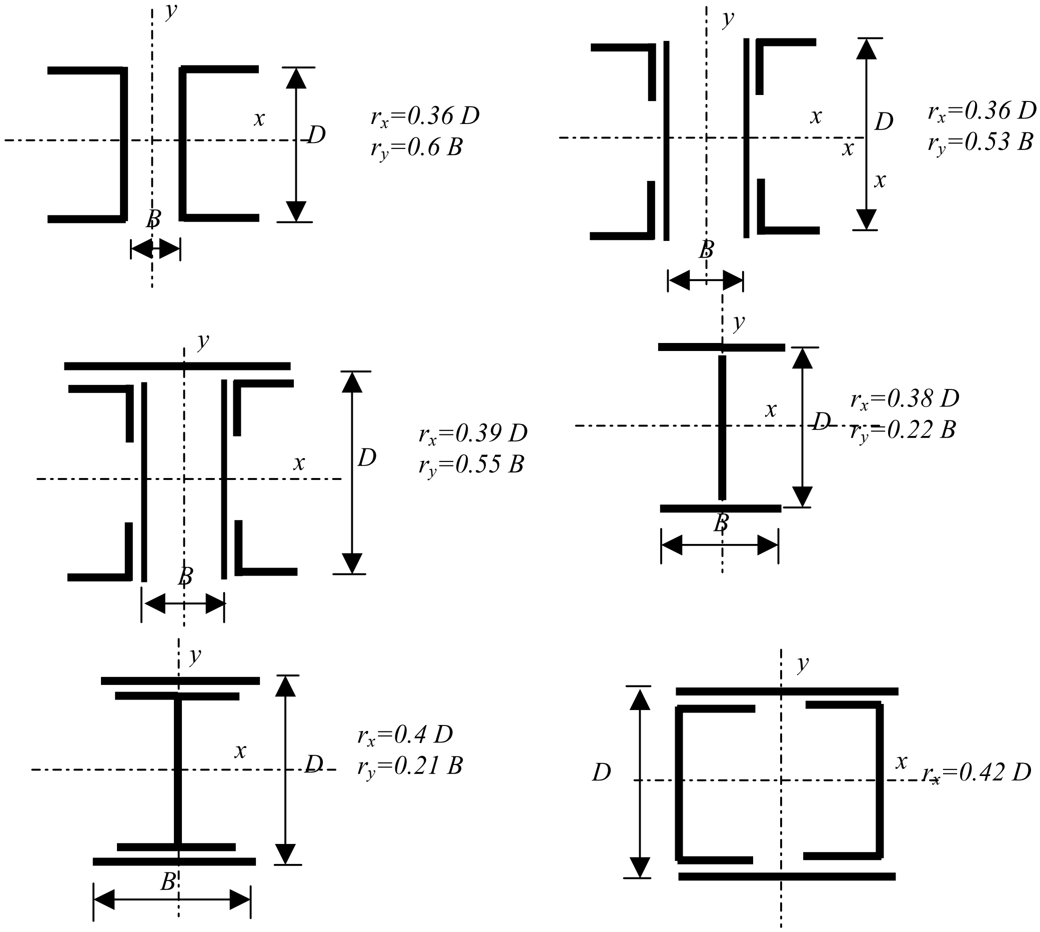


Fig 8: Approximate radii of gyration

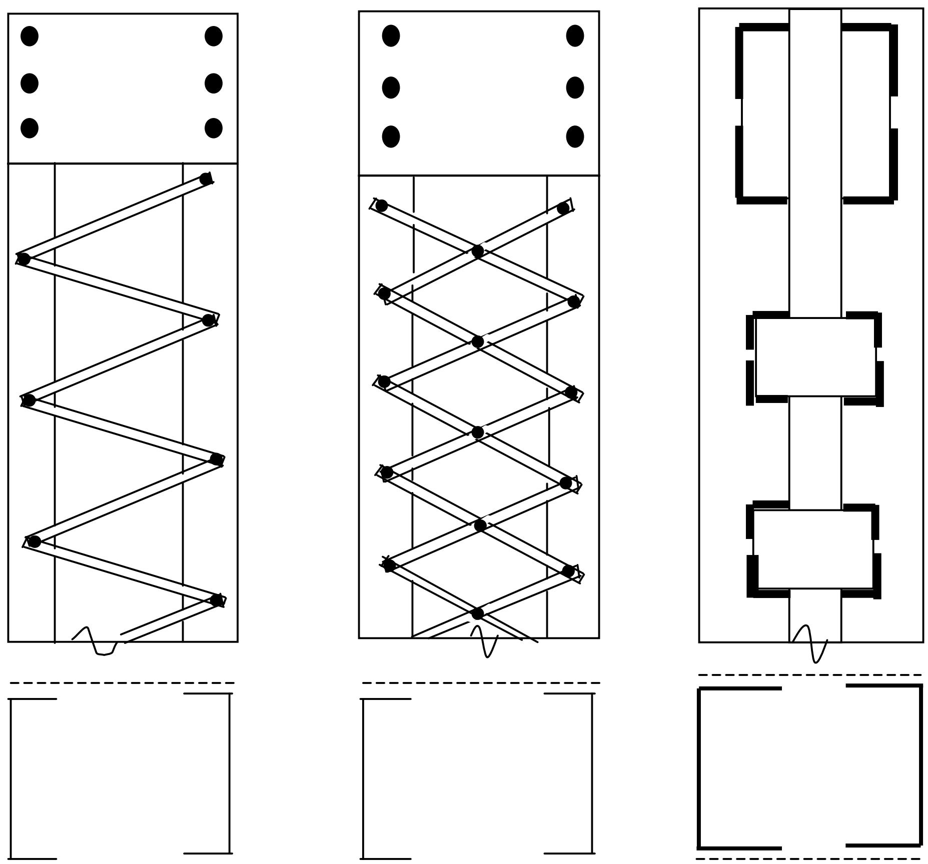
Open Web Columns:

In Fig. 9 the two channel sections of the column are connected together by batten plates or laces which are shown by dotted lines. A typical lacing or batten plate is shown in Fig. 9. Laced columns (also called latticed columns) generally carry *10%* more load than battened columns for the same area of cross section. This necessitates a *10%* increase in the slenderness ratio for battened columns. All columns should be tied at the ends by tie plates or end battens to ensure a satisfactory performance.

6.3 Design Considerations for Laced and Battened Columns

The two channel constituents of a laced column, shown in Fig. 9*(a)* and 9 *(b)* have a tendency to buckle independently. Lacing provides a tying force to ensure that the channels do not do so. The load that these tying forces cause is generally assumed to cause a shearing force equal to *2.5%* of axial load on the column. (Additionally if the columns are subjected to moments or lateral loading the lacing should be designed for the additional bending moment and shear). To prevent local buckling of unsupported lengths between the two constituent lattice points (or between two battens), the slenderness ratio of individual components should be less than *50* % or *70%* of the slenderness ratio of the built up column (whichever is less).

In laced columns, the lacing should be symmetrical in any two opposing faces to avoid torsion. Lacings and battens are not combined in the same column. The inclination of lacing bars from the axis of the column should not be less than 40^0 nor more than 70^0. The slenderness ratio of the lacing bars should not exceed *145*. The effective length of lacing bars is the length between bolts for single lacing and *0.7* of this length for double lacing. The width of the lacing bar should be at least *3* times the diameter of the bolt. Thickness of lacing bars should be at least $1/40^{th}$ of the length between bolts for single lacing and *1/60* of this length for double lacing (both for welded and bolted connections).



(a) Single Lacing *(b) Double Lacing* *(c) Battens*

Fig. 9 Built-up column members

In the Western world, it was common practice to "build-up" the required cross sectional area of steel compression members from a number of smaller sections. Since then, the increasing availability of larger rolled steel sections and the high fabrication costs have resulted in a very large drop in the use of built-up compression members. The disrepute of built-up compression members arises from the unrealistic expectation of many designers that a built-up member should have the same capacity of a solid member and also behaves in every respect in an identical manner to the latter. Early designers were disappointed to discover that this was just not possible. A contributory cause to this decline was the restrictive clauses introduced in many western design codes, following the Quebec Bridge Failure in *1907*. However there is a continuing use of built-up members, where stiffness and lightness are required, as in Transmission line Towers. There is, however, a wide spread use of built up compression members in the developing world, India included, largely because of the non-availability of heavier rolled sections and the perception that fabrication of members is cheaper.

For Laced and Battened Columns, Section 7.6 and 7.7 of IS: 800 shall be followed. In practical columns, the battens are very stiff and as they are normally welded to the vertical members, they can be considered as rigid connectors.

To allow approximately for this behaviour, a modified formula for calculating the effective slenderness (λ_b) of battened columns has been widely employed. This ensures that the Perry-Robertson approach outlined earlier could be used with a modified value for slenderness given by λ_b defined below.

$$\lambda_b = \sqrt{\lambda_f^2 + \lambda^2} \qquad (7)$$

where,

λ_f = lower value of slenderness of the individual vertical members between batten intervals and

λ = slenderness of the overall column, using the radius of gyration of the whole built up section.

This equation, though an approximation, has been shown by Porter and Williams of Cardiff University actually to give accurate and safe values over the entire range of practical parameters for uniform columns with normal depth battens. In calculating the values of σ_e in the Perry Robertson equation, [equation 1 and 1(a)] λ_b is to be employed in place of λ, using slenderness ratio in defined in equation (3). The imperfection (η_b) is calculated from

$$\eta_b = 0.0055(\lambda_b) \qquad (8)$$

The strength of the battened column is evaluated from

$$\sigma_c = \frac{f_y + (\eta_b + 1)\sigma_e}{2} - \sqrt{\left[\frac{f_y + (\eta_b + 1)\sigma_e}{2}\right]^2 - f_y \cdot \sigma_e} \qquad (9)$$

λ_b = effective slenderness with η_b computed as given in equation (8)

σ_e = calculated using η_b values given in equation (7)

7.0 BASE PLATES FOR CONCENTRICALLY LOADED COLUMNS

The design compressive stress in a concrete footing is much smaller than it is in a steel column. So it becomes necessary that a suitable base plate should be provided below the column to distribute the load from it evenly to the footing below. The main function of the base plate is to spread the column load over a sufficiently wide area and keep the footing from being over stressed.

For a purely axial load, a plain square steel plate or a slab attached to the column is adequate. If uplift or overturning forces are present, a more positive attachment is necessary. These base plates can be welded directly to the columns or they can be fastened by means of bolted or welded lug angles. These connection methods are illustrated in Fig. 10.

A base plate welded directly to the columns is shown in Fig. 10(*a*). For small columns these plates will be shop-welded to the columns, but for larger columns, it may be necessary to ship the plates separately and set them to the correct elevations. For this second case the columns are connected to the footing with anchor bolts that pass through the lug angles which have been shop-welded to the columns. This type of arrangement is shown in Fig. 10(*b*).

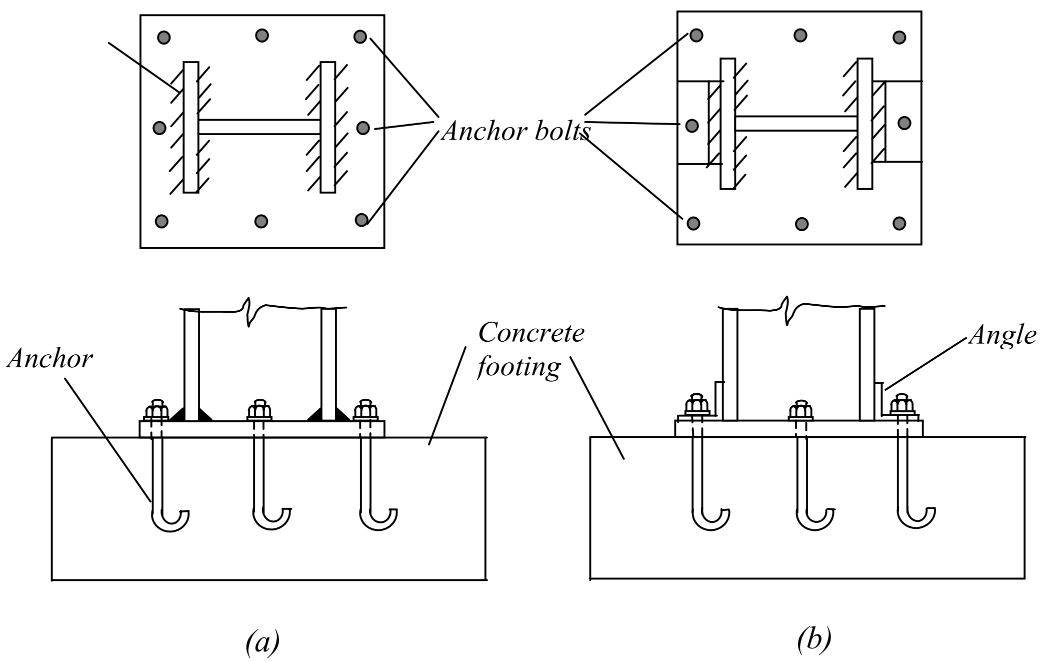


Fig. 10 Column base

Sometimes, when there is a large moment in relation to the vertically applied load a gusseted base may be required. This is intended to allow the lever arm from the holding down bolts to be increased to give maximum efficiency while keeping the base plate thickness to an acceptable minimum.

A critical phase in the erection of a steel building is the proper positioning of column base plates. If they are not located at their correct elevations, serious stress changes may occur in the beams and columns of the steel frame. In many cases, levelling plates of the same dimensions as the base plate are carefully grouted in place to the proper elevations first and then the columns with attached base plates are set on the levelling plates.
The lengths and widths of column base plates are usually selected in multiples of *10 mm* and the thickness chosen to conform to rolled steel plates. Usually the thickness of base plates is in the range of *40-50 mm*. If plates of this range are insufficient to develop the applied bending moment or if thinner plates are used, some form of stiffening must be provided.

Concrete support area should be significantly larger than the base plate area so that the applied load can disperse satisfactorily on to the foundation. To spread the column loads uniformly over the base plates, and to ensure there is good contact between the two, it is customary not to polish the underside of the base plate, but grout it in place.

Columns supporting predominantly axial loads are designed as being pin-ended at the base. The design steps for a base plate attached to an axially loaded column with pinned base is explained below.

Procedure for empirical design of a slab base plate for axial load only (pinned connection)

1. Determine the factored axial load and shear at the column base.
2. Decide on the number and type of holding down bolts to resist shear and tension. The chosen number of bolts are to be arranged symmetrically near corners of base plate or next to column web, similar to the arrangement sketched in Fig. 10.
3. Maximum allowable bearing strength $= 0.4 f_{cu}$ (where f_{cu} = cube strength of concrete) Actual bearing pressure to be less than or equal to $0.4 f_{cu}$.
4. Determine base plate thickness t;

For *I, H*, *channel, box* or *RHS columns*

$t = \sqrt{\frac{2.5w}{f_{yp}}\left(a^2 - 0.3b^2\right)}$ but not less than the thickness of the flange of the supported column.

w = pressure in N/mm^2 on underside of plate, assuming a uniform distribution.
a = larger plate projection from column [See Fig. 11]
b = smaller plate projection from column
f_{yp} = design strength of plate, but not greater than $250\ N/mm^2$ divided by γ_m

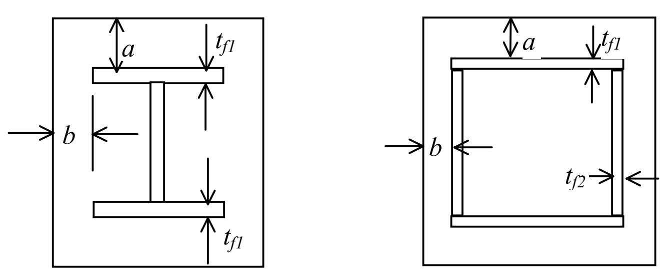


Fig. 11 Base plates subjected to concentric forces

5. Check for adequacy of weld. Calculate the total length of weld to resist axial load.
6. Select weld size.
7. Check shear stress on weld.
8. Vector sum of all the stresses carried by the weld must not exceed $p_{w,}$ the design strength, of the weld.
9. Check for bolt. Check maximum co-existent factored shear and tension, if any, on the holding down bolts.
10. Check the bolts for adequacy (see a later chapter for bolt design).

Stipulations of IS: 800 for Column Bases

Column bases should have sufficient, stiffness and strength to transmit axial force, bending moments and shear forces at the base of the columns to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should be provided wherever necessary. Shear resistance at the proper contact surface between steel base and concrete/grout may be calculated using a friction coefficient of 0.45.

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure should not exceed the bearing strength equal to $0.6f_{ck}$, where f_{ck} is the smaller of characteristic cube strength of concrete or bedding material.

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, *c*, of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load (Fig 7.2), such that beam pressure as the effective area does not exceed bearing capacity of concrete base.

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

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

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

Slab Bases – Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection.

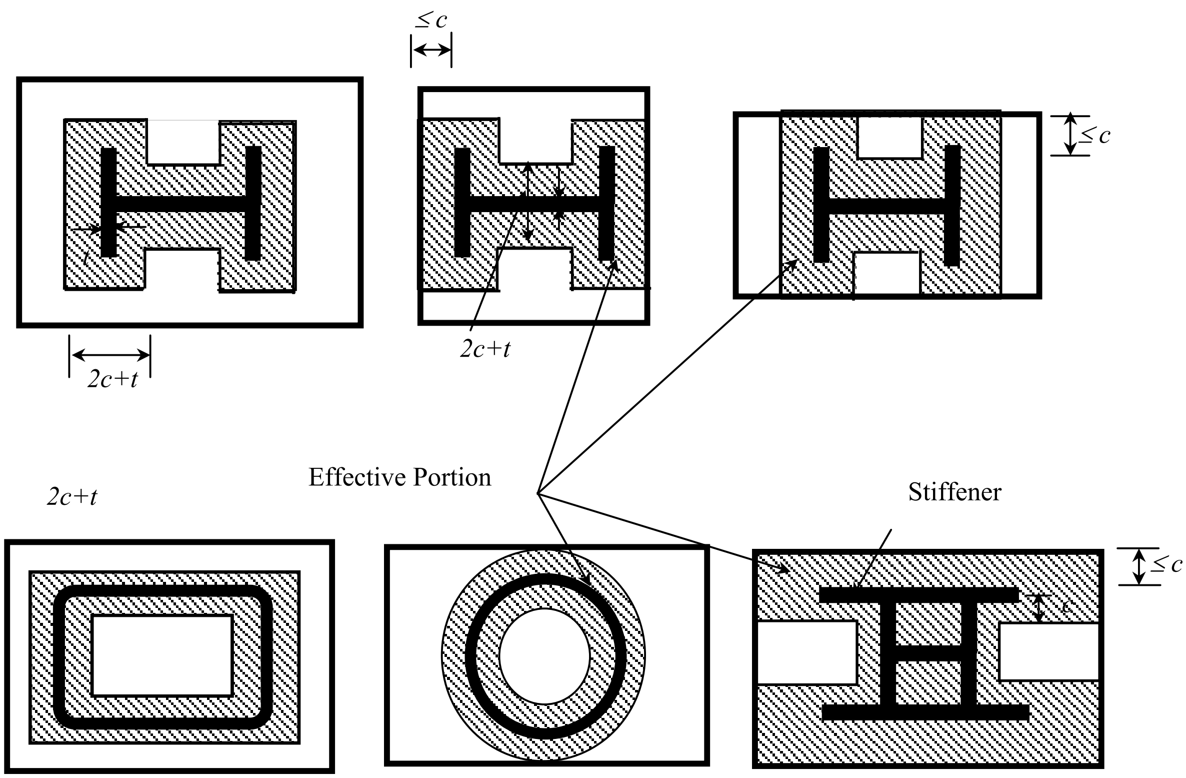


FIG 7.2 EFFECTIVE AREA OF A BASE PLATE

The minimum thickness, t_s, of rectangular slab bases, supporting columns under axial compression shall be

$$t_s = \sqrt{2.5\,w(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > \; t_f$$

where,

w = uniform pressure from below on the slab base under the factored load axial compression

a, b = larger and smaller projection of the slab base beyond the rectangle circumscribing the column, respectively

t_f = flange thickness of compression member

When only the effective area of the base plate is used as in section 7.4.1.1instead of $(a^2 - 0.3b^2)$, c^2 may be used in the above equation (see Fig 7.2)

When the slab does not distribute the column load uniformly, due to eccentricity of the load etc, special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

Bases for bearing upon concrete or masonry need not be machined on the underside.

In cases where the cap or base is fillet welded directly to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required in **7.4.3**. Where full strength butt welds are provided, machining of contact surfaces is not required.

7.0 CONCLUDING REMARKS

Design of columns using multiple column curves is discussed in this chapter. Additional provision required for accounting for heavily welded sections are detailed. Built-up fabricated members frequently employed (when rolled sections are found inadequate) are discussed in detail. Design guidance is provided for laced/battened columns. Effective lengths for various end conditions are listed and illustrative worked examples are appended. A simple method of designing a base plate for an axially loaded column is proposed and illustrated by a worked example. Wherever required, provisions of IS: 800 (latest Version) have been highlighted and worked examples have been prepared based on the stipulations of IS: 800 (Latest Version).

8.0 REFERENCES

1. Owens G.W., Knowles P.R (1994): "Steel Designers Manual", The Steel Construction Institute, Ascot, England.
2. Dowling P.J., Knowles P.R., Owens G.W (1998): "Structural Steel Design", Butterworths, London.
3. British Standards Institution (1985): "BS 5950, Part-1 Structural use of steelwork in building", British Standards Institution, London.
4. Bureau of Indian Standard, IS: 800,

Table3: Ultimate Compressive stress (σ_c) values in compression members (f_y = 250 N/mm^2)

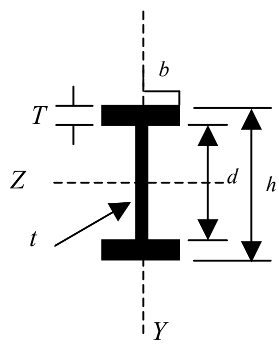
λ	$\alpha = 0.002$	$\alpha = 0.0035$	$\alpha = 0.0055$	$\alpha = 0.008$
15	250	250	250	250
20	249	248	247	245
25	246	243	240	235
30	243	239	233	225
35	240	234	225	216
40	237	228	218	206
45	233	223	210	196
50	229	216	202	187
55	225	210	194	177
60	219	203	185	168
65	213	195	176	160
70	206	187	168	150
75	198	178	159	141
80	189	169	150	133
85	180	160	141	125
90	170	151	133	118
95	159	142	125	111
100	149	133	118	104
110	130	117	104	92
120	114	103	92	82
130	99	91	82	73
140	87	80	73	66
150	77	71	65	59
160	69	64	59	53
170	62	57	53	48
180	55	52	48	44
190	55	47	44	40
200	45	43	40	37

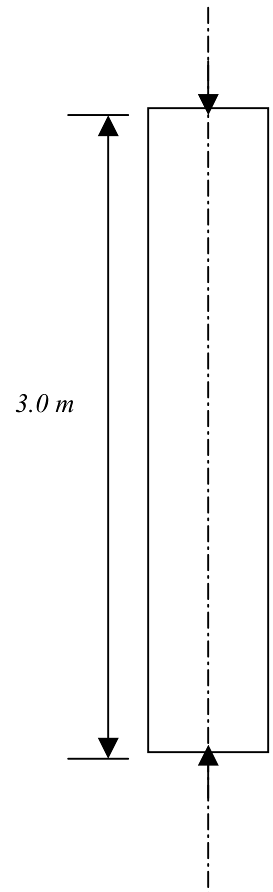
Structural Steel Design Project Calculation Sheet	Job No:	Sheet 1 of 4	Rev
	Job Title: AXIALLY COMPRESSED COLUMN		
	Worked Example - 1		
		Made by GC	Date 23-02-07
		Checked by TKB	Date 28-02-07

Obtain factored axial load on the column section ISHB400. The height of the column is 3.0m and it is pin-ended.

[$f_y = 250\ N/mm^2$; $E = 2\ x10^5\ N/mm^2$; $\gamma_m = 1.10$] — *Table 5.2 of IS: 800*

CROSS-SECTION PROPERTIES:





Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet **2 of 4**	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *GC*	Date *23-02-07*
		Checked by *TKB*	Date *28-02-07*

Flange thickness = T = *12.7 mm*

Overall height of ISHB400 = h = *400 mm*

Clear depth between flanges = d = *400 – (12.7 * 2) = 374.6 mm*

Thickness of web = t = *10.6 mm*

Flange width = $2b$ = b_f = *250 mm*

Hence, half Flange Width = b = *125 mm*

Self –weight = w = *0.822 kN/m*

Area of cross-section = A = *10466 mm*^{2}

Radius of gyration about x = r_x = *166.1 mm*

Radius of gyration about y = r_y = *51.6 mm*

(i) Type of section:

$$\frac{b}{T} = \frac{125}{12.7} = 9.8 < 10.5\varepsilon$$

$$\frac{d}{t} = \frac{374.6}{10.6} = 35.3 < 42\varepsilon$$

Table 3.1 of IS: 800

$$where, \varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Hence, cross- section can be classified as "COMPACT"

(ii) Effective Sectional Area, A_e = *10466 mm*^{2}

(Since there is no hole, no reduction has been considered)

Clause 7.3.2 of IS: 800

Structural Steel Design Project	Job No:	Sheet ***3 of 4***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *GC*	Date *23-02-07*
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(iii) Effective Length:

As, both ends are pin-jointed effective length $KL_x = KL_y = 1.0 \times L_x = 1.0 \times L_y = 1.0 \times 3.0\ m = 3.0\ m$

Clause 7.2 and Table 7.5 of IS: 800

(iv) Slenderness ratios:

$$KL_x / r_x = \frac{3000}{166.1} = 18.1$$

$$KL_y / r_y = \frac{3000}{51.6} = 58.1$$

(v) Non-dimensional Effective Slenderness ratio, λ :

$$\lambda = \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E} = \sqrt{250x(58.1)^2 / \pi^2 x2x10^5}$$

$$= \mathbf{0.654}$$

Clause 7.1.2.1 of IS: 800

(vi) Value of ϕ from equation $\phi = 0.5[1+\alpha(\lambda - 0.2) + \lambda^2]$:

Where, α = Imperfection Factor which depends on Buckling Class
Now, from Table 7.2 of Chapter 7, for $h/b_f = 400 / 250 = 1.6 > 1.2$ *and also thickness of flange, T = 12.7 mm, hence for z-z axis buckling class 'a' and for y-y axis buckling class 'b' will be followed.*

Table 7.1 of IS: 800

Hence, $\alpha = 0.34$ for buckling class 'b' will be considered.

Hence, $\boldsymbol{\phi = 0.5 \times [1+0.34 \times (0.654-0.2)+0.654^2] = 0.791}$

(vii) Calculation of χ from equation $\chi = \left[1 \Big/ \left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right] \right]$:

$$\chi = \left[1 \Big/ \left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5}\right] \right] = \left[1 \Big/ \left[0.791 + \left(0.791^2 - 0.654^2\right)^{0.5}\right] \right]$$

$$= \mathbf{0.809}$$

Structural Steel Design Project	Job No:	Sheet *4 of 4*	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *GC*	Date *23-02-07*
Calculation Sheet		Checked by *TKB*	Date *28-02-07*

(viii) Calculation of f_{cd} from the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} = 0.809 x 250 / 1.10 = 183.86 \text{ N/mm}^2$$

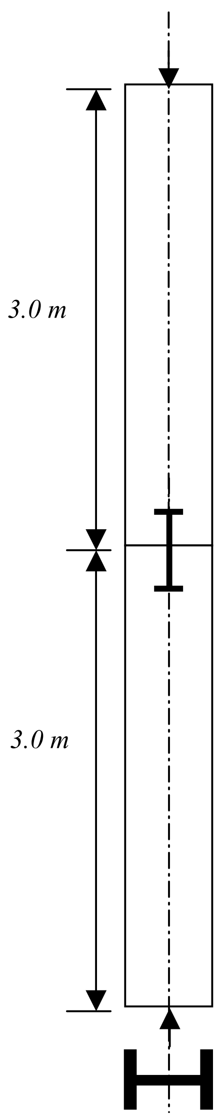
(ix) Factored Axial Load in kN, P_d:

$P_d = A_e f_{cd} = 10466 \times 183.86/1000 = 1924.28$ kN

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 2		
		Made by *GC*	Date *23-02-07*
Calculation Sheet		Checked by *TKB*	Date *28-02-07*

Obtain maximum axial load carried by the column shown when ISHB 400 is employed. The column is effectively restrained at mid-height in the y-direction, but is free in x-axis. The data is the same as in problem1.

[$f_y = 250\ N/mm^2$; $E = 2.0 \times 10^5\ N/mm^2$; $\gamma_m = 1.10$]



(i) Type of section:

Section is "COMPACT" from previous example.

(ii) Effective lengths:

As, both ends are pin-jointed effective length $KL_x = 1.0 \times 6000 = 6000$ mm

$KL_y = 1.0 \times 3000 = 3000$ mm

(iii) Slenderness ratios:

$$KL_x / r_x = \frac{6000}{166.1} = 36.12$$

$$KL_y / r_y = \frac{3000}{51.6} = 58.1$$

(iv) Non-dimensional Effective Slenderness ratio, λ :

As calculated in previous example,

$\lambda = \mathbf{0.654}$ (for worst slenderness ratio)

(v) Value of ϕ:

As calculated in previous example,

$\phi = \mathbf{0.791}$ (for $\lambda = \mathbf{0.654}$)

(vi) Value of χ:

As calculated in previous example,

$\chi = \mathbf{0.809}$ (for $\phi = \mathbf{0.791}$)

Structural Steel Design Project	Job No:	Sheet *2 of 2*	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 2		
		Made by *GC*	Date *23-02-07*
Calculation Sheet		Checked by *TKB*	Date *28-02-07*

(vii) Calculation of f_{cd}:

As calculated in previous example,

f_{cd} = ***183.86*** N/mm^2

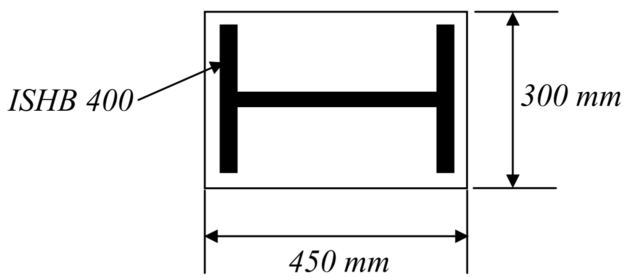
(viii) Calculation of Factored Load:

Factored Load = $f_{cd} \, x A_e/\gamma_m$ = 183.86/1.10 x10466/1000= 1924.28 kN

Structural Steel Design Project	Job No:	Sheet ***1 of 1***	Rev
	Job Title: *BASE PLATE*		
	Worked Example - 3		
		Made by *GC*	Date *23-02-07*
Calculation Sheet		Checked by *TKB*	Date *28-02-07*

Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN.

[f_{cu} = 40 N/mm^2 ; f_y = 250 N/mm^2 ; γ_m = 1.10] — *Table 5.2 of IS: 800*



Thickness of Flange for ISHB400 = T = 12.7 mm

*Bearing strength of concrete = 0.4f_{cu} = 0.4 * 40 = 16 N/mm^2*

*Area required = 1800*10^3/16 = 112500 mm^2*

Use plate of 450 X 300 mm (135000 mm^2)

Assuming projection of 25 mm on each side, a = b = 25 mm

$$w = \left(1800 \times 10^3 / 450 \times 300\right) = 13.33 \; N/mm^2$$

Now thickness of Slab Base, t_s

$$t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > \quad T$$

Clause 7.4.3.1 of IS: 800

$$= \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)\times 1.10}{f_y}} = \sqrt{\frac{2.5\times 13.33\times\left(25^2 - 0.3\times 25^2\right)\times 1.10}{250}} = 8.01\ mm$$

< T = 12.7 mm, Hence provide a base plate of thickness not less than 12.7 mm and since the available next higher thickness of plate is 16 mm

Use 450 X 300 X 16 mm plate.