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 (ℓ/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.

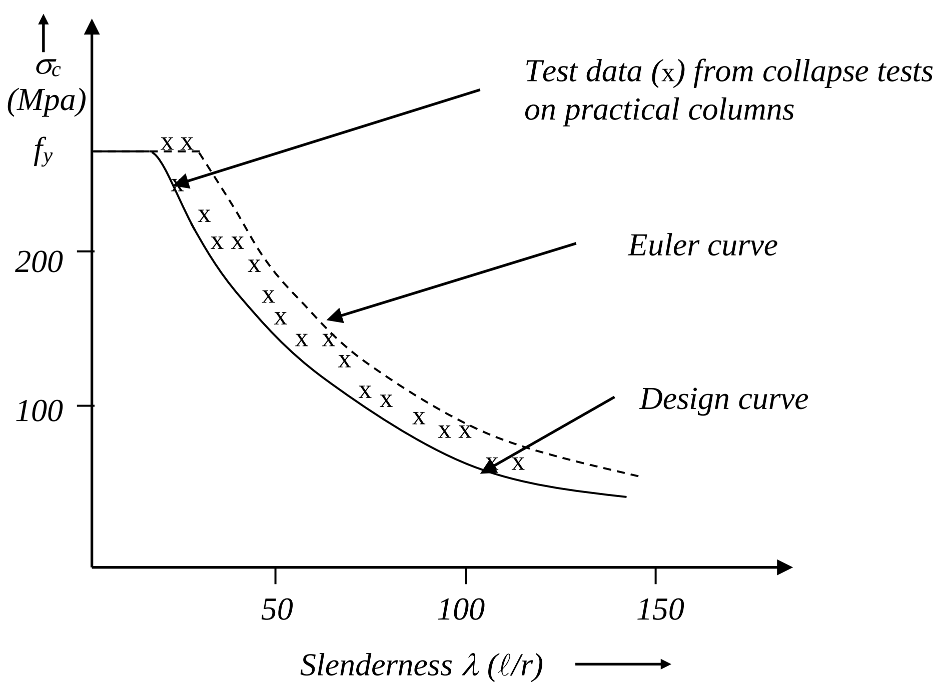

Fig. 1 Typical column design curve

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.

$$(f_y - \sigma_c)\ (\sigma_e - \sigma_c) = \eta \cdot \sigma_e \cdot \sigma_c \qquad (1)$$

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 (ℓ/r)

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

$$\eta = \frac{y \cdot \Delta}{r^2} \qquad (2)$$

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

$$\eta = \left(0.001\frac{y}{r}\right) \cdot \left(\frac{\ell}{r}\right)$$

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

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

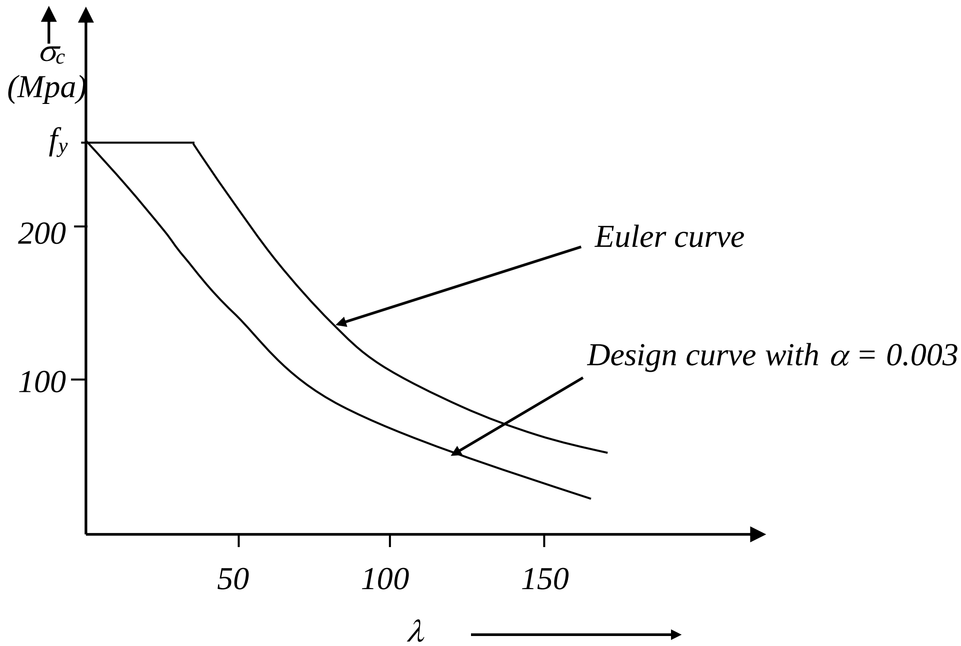

Fig.2 Robertson's Design Curve

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 $\lambda \le \lambda_e$ as the column would fail by squashing at f_y.

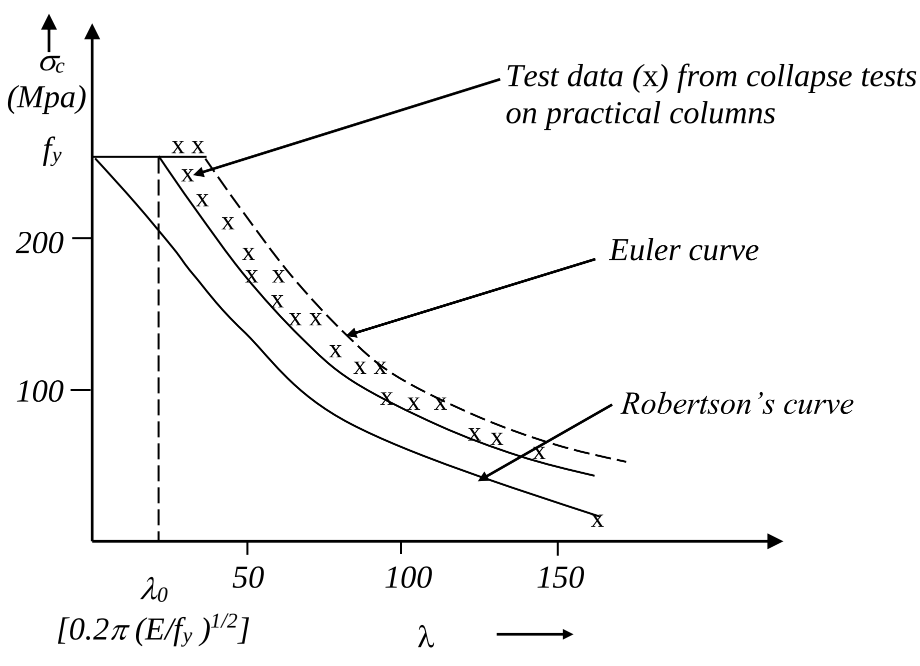

Fig.3 Strut curve with a plateau for low slenderness values

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*)].

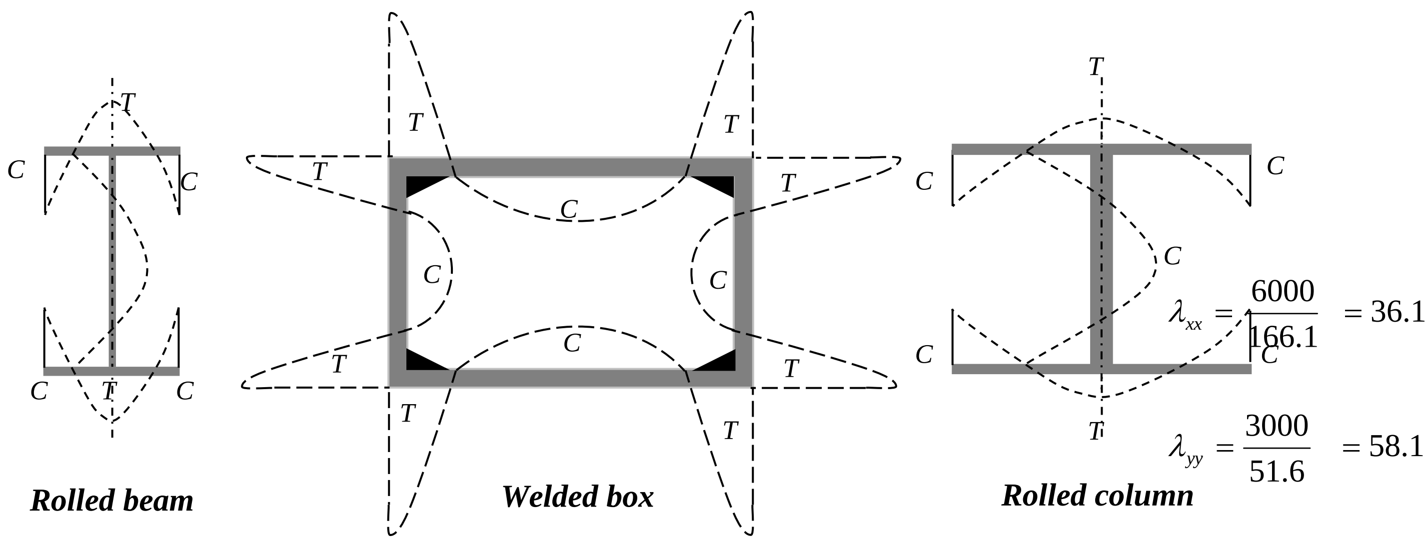

Fig. 4 Distribution of residual stresses

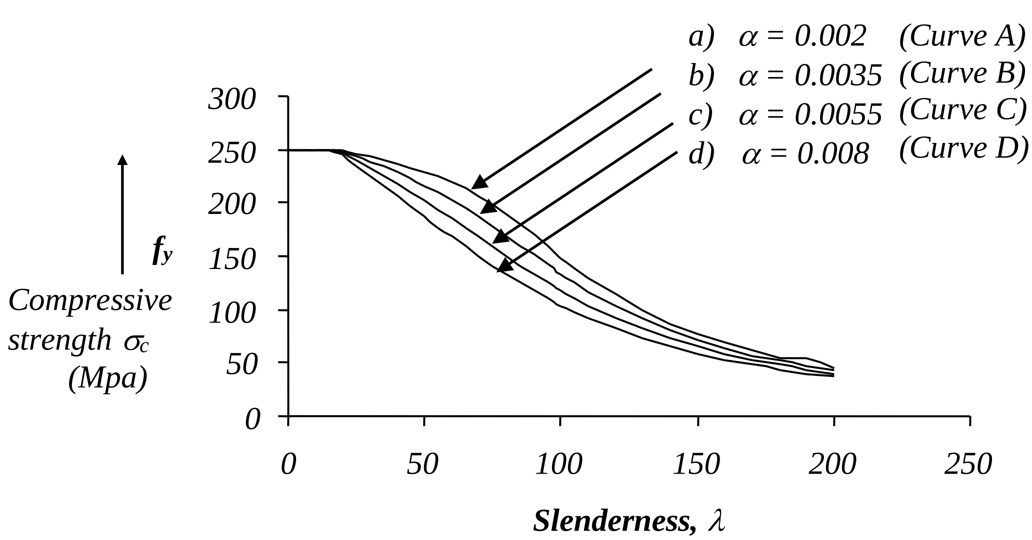

Fig. 5(a) Compressive strength curves for struts for different values of α

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

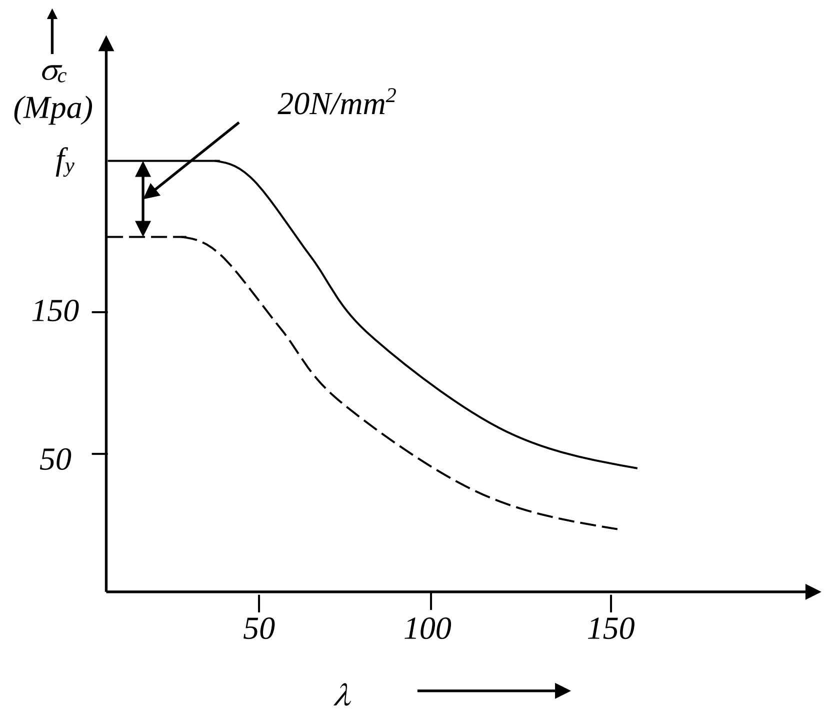

Fig. 5(b) Compressive strength of welded sections

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.

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

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

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

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 *X*, *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. ***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)***. Values of α to be used in all these cases are available in published codes and are typically *0.002*, *0.0035*, *0.0055* and *0.008* depending on the type of the column section and the extent of residual stresses present. The values suggested in British Codes are included in Fig. 5(*a*) and Table 3 gives the maximum axial compressive stresses using these α values, for f_y equals to *250 M pa*.

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. The British Code recommends that the design strength values be reduced by *20 N/mm^2* and the resulting curve would be adequate to account for the reduced buckling capacity [See Fig. 5(*b*)]

4.0 EFFECTIVE LENGTH OF COLUMNS

Pin-ended axially loaded columns are rare in practice. However, available analytical studies – starting from Euler's Theory – are all based on the assumption of pin-ended columns. Results of experiments carried out on pin ended struts for over *100* years are well documented in published literature. Hence we employ the columns hinged at both ends as the standard case for purposes of comparison and the concept of "effective lengths" for columns having various boundary conditions was presented in a previous chapter. By employing the effective lengths (ℓ_e) appropriate to the column under consideration, it is usually possible to predict its strength. ℓ_e may regarded as the equivalent length of a pin-ended column having the same cross section, which would be expected to have the same strength and stiffness as the column being designed.

Ideally fixed ended columns are difficult to achieve in practice. BS: 5950 Part-1 recommends the following effective lengths for design purposes

Column pin-ended at both ends	ℓ_e	=	1.0ℓ
Column pin-ended at one end and fixed at the other	ℓ_e	=	0.85ℓ
Column fixed at both ends	ℓ_e	=	$0.7\ \ell$
Column fixed at one end and on roller support at the other	ℓ_e	=	$1.2\ \ell$
Column fixed at one end and free at the other	ℓ_e	=	$2.0\ \ell$

(Note: These values are NOT the same as those contained in IS: 800-1984. For the second and third boundary conditions IS: 800-1984 suggests *0.80* ℓ and *0.65* ℓ respectively which are also followed by Australian and American practices)

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 THE DESIGN OF AXIALLY LOADED COLUMNS

The procedure for the design of an axially compressed column is as follows:

(i) Assume a suitable trial section and classify the section in accordance with the classification in "Local Buckling and Section classification" chapter. (If, the section is slender then apply appropriate correction factor)

(ii) Arrive at the effective length of the column by suitably considering the end conditions.

(iii) Calculate the slenderness ratios (λ values) in both minor and major axes direction and also calculate λ_0 using the formula given below:

$$\lambda_0 = 0.2\pi\sqrt{E/f_y}$$

(iv) Calculate σ_e values along both major and minor axes from equation 1(a)

(v) Calculate $\eta = [(\lambda - \lambda_0)\alpha]$, using appropriate α values from Table 2

(vi) Calculate ϕ from equation (6a) and σ_c from equation (5)

(vii) Compute the load that the compression member can resist $(\sigma_c A)$

(viii) 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 (ℓ/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.

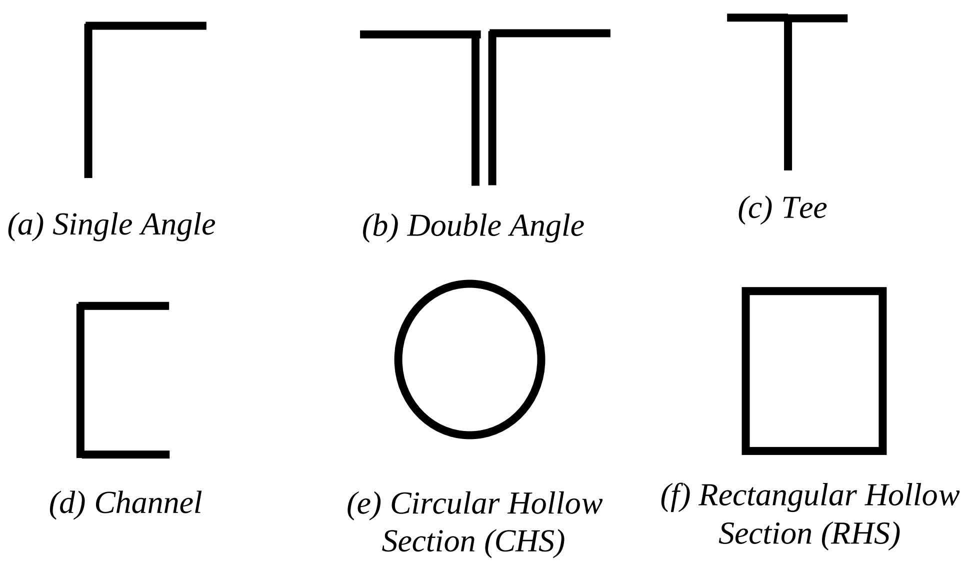

Fig 6: Cross Section Shapes for Rolled Steel Compression Members

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 $\left(\ell/r \approx 150\right)$

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

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

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.

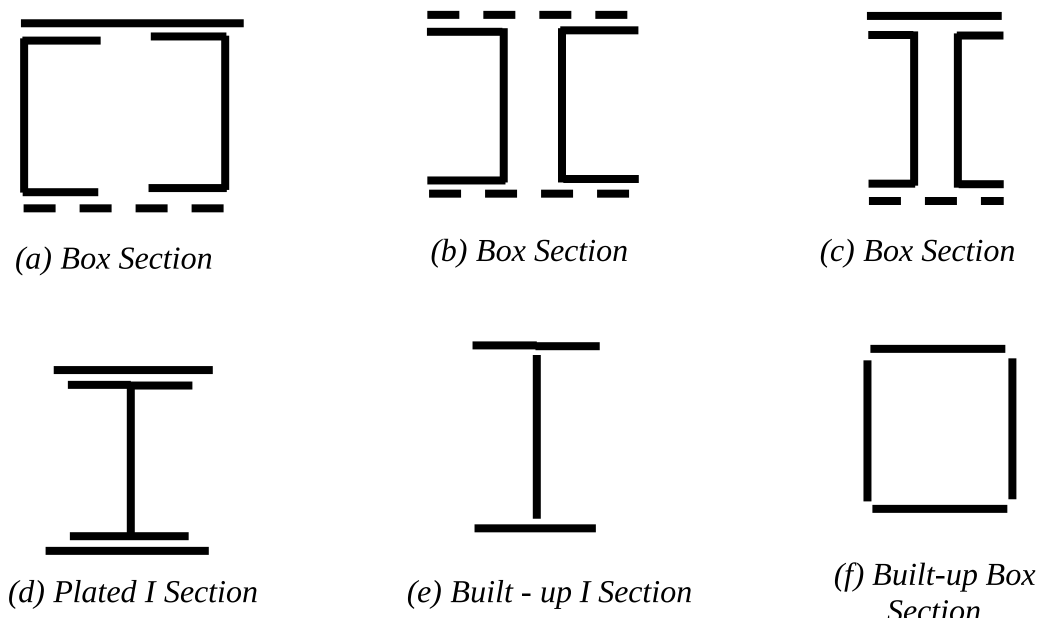

Fig 7: Cross Section Shapes for Built - up or fabricated Compression Members

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).

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.

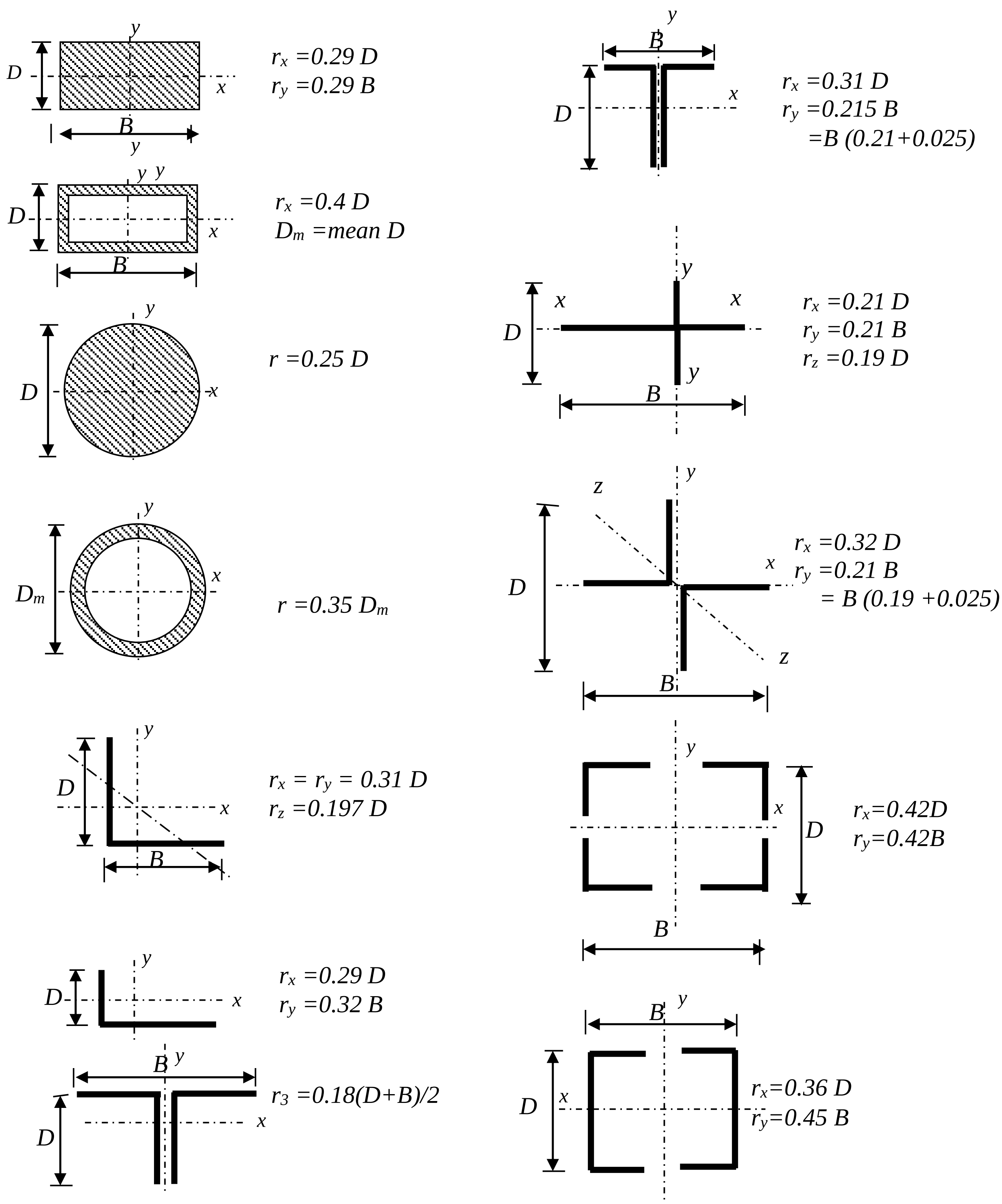

Fig 8: Approximate radii of gyration
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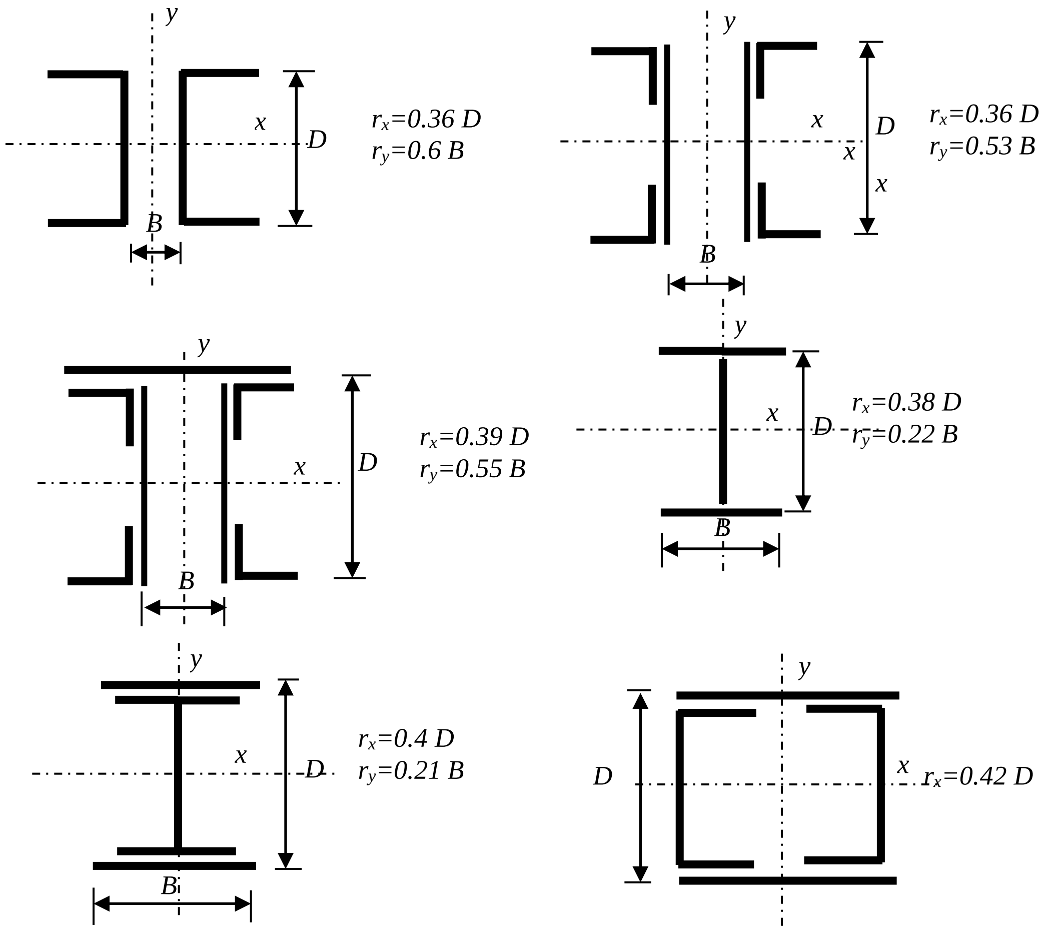
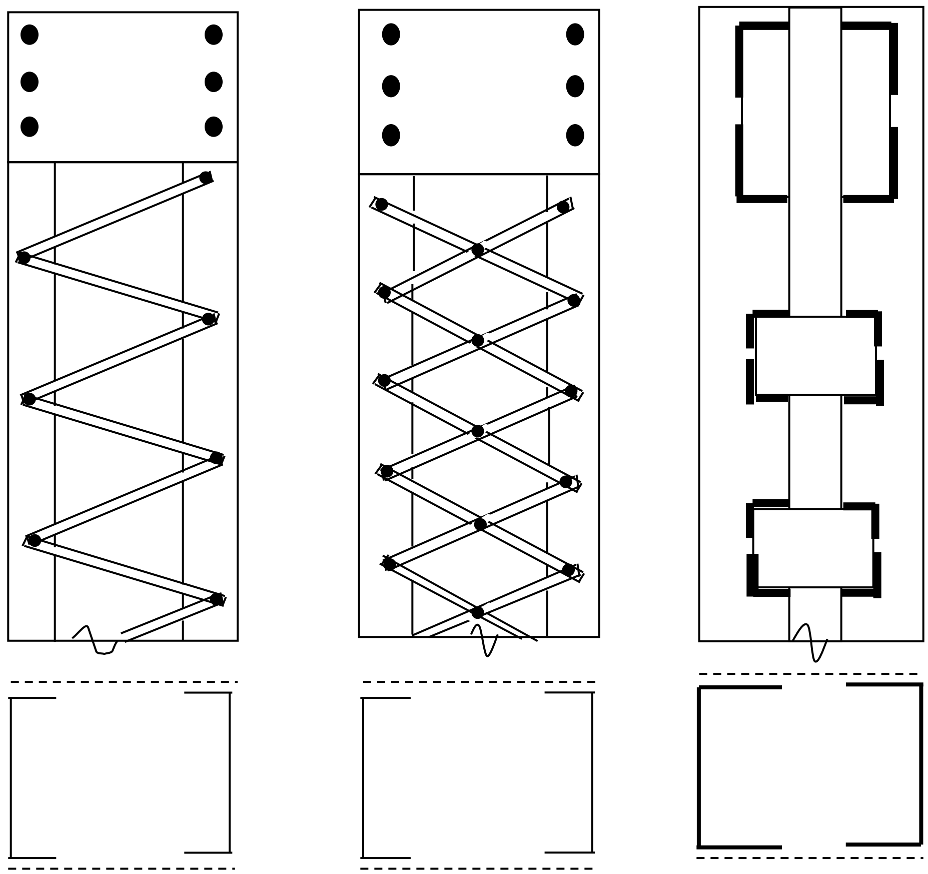


Fig 8: Approximate radii of gyration

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 *1/40th* 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.

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.

$$\lambda_b = \sqrt{\lambda_f^2 + \lambda^2} \tag{7}$$

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

$$\eta_b = 0.0055(\lambda_b) \tag{8}$$

The strength of the battened column is evaluated from

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

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*).

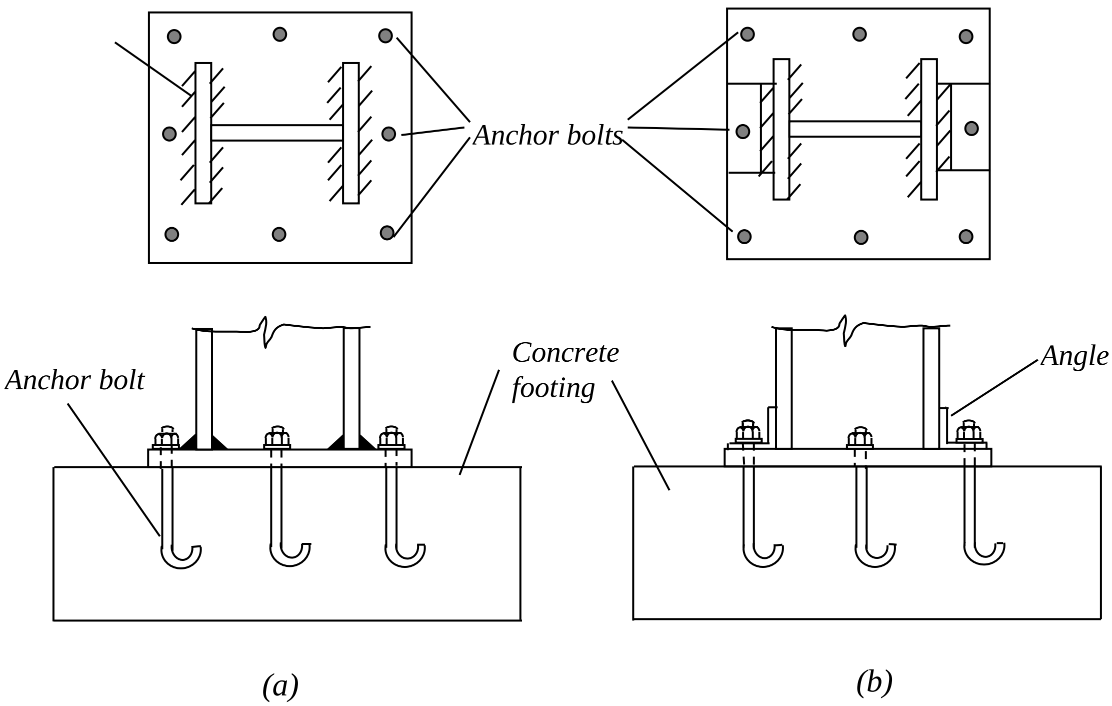


Fig. 10 Column base plates

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*

$t = \sqrt{\dfrac{2.5w}{f_{yp}}\left(a^2 - 0.3b^2\right)}$ 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

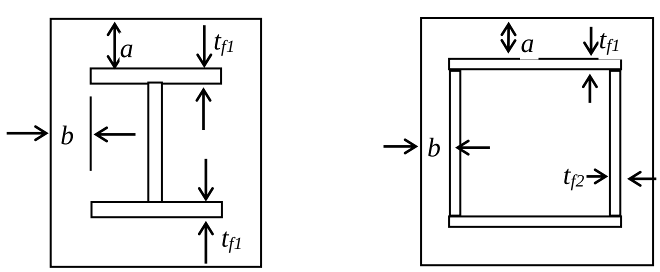


Fig. 11 Base plates subjected to concentric forces

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).

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.

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.

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

λ	α = 0.002	α = 0.0035	α = 0.0055	α = 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

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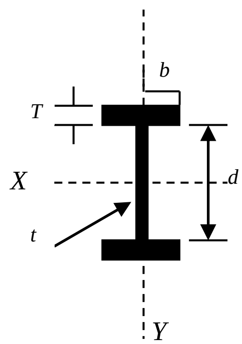
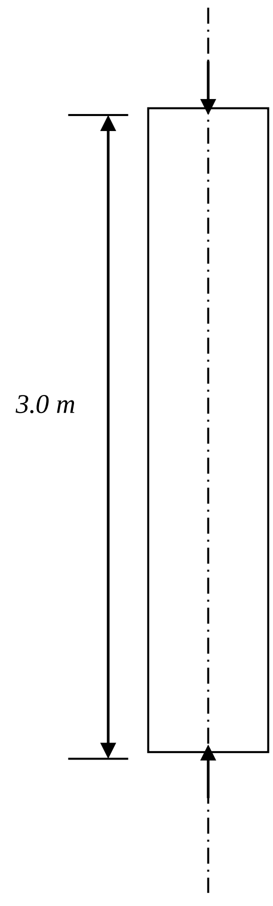
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date 2*3-09-99*
Calculation Sheet		Checked by *RN*	Date 2*8-09-99*

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

[$f_y = 250\ N/mm^2$; $E = 2*10^5\ N/mm^2$; $\gamma_m = 1.15$]

CROSS-SECTION PROPERTIES:





<table>
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<tr><td colspan="3">Job Title: AXIALLY COMPRESSED COLUMN</td></tr>
<tr><td colspan="3">Worked Example - 1</td></tr>
<tr><td rowspan="2"></td><td>Made by SSSR</td><td>Date 23-09-99</td></tr>
<tr><td>Checked by RN</td><td>Date 28-09-99</td></tr>
</table>

Flange thickness = T = *12.7 mm*

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

Thickness of web = t = *10.6 mm*

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

b = *125 mm*

Self –weight = w = *0.822 kN/m*

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

r_x = *166.1 mm*

r_y = *51.6 mm*

(i) Type of section:

$$\frac{b}{T} = \frac{125}{12.7} = 9.8 < 10 \in$$

$$\frac{d}{t} = \frac{374.6}{10.6} = 35.3 < 41 \in$$

BS: 5950
Part - 1
Table - 7

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

Hence, cross- section is "COMPACT"

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	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
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(ii) ***Effective Length:***

As, both ends are pin-jointed effective length = $\ell_x = \ell_y = 3.0$ *m*

(iii) ***Slenderness ratios:***

$$\lambda_x = \frac{\ell_x}{r_x} = \frac{3000}{166.1} = 18.1$$

$$\lambda_y = \frac{\ell_y}{r_y} = \frac{3000}{51.6} = 58.1$$

(iv) ***Values of*** (α) **:**

For rolled I-sections,

In x – direction $\alpha_x = 0.0020$
In y – direction $\alpha_y = 0.0035$

$$\lambda_o = 0.2\pi\sqrt{E/f_y} = 0.2*\frac{22}{7}\sqrt{\frac{200000}{250}}$$

$$= 17.8$$

(v) ***values of*** η**:**

$$\eta = \alpha(\lambda - \lambda_0)$$

$$\eta_x = \alpha_x(\lambda_x - \lambda_0) = 0.002*(18.1 - 17.8) = 0.001$$

$$\eta_y = \alpha_y(\lambda_y - \lambda_0) = 0.0035*(58.1 - 17.8) = 0.141$$

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	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
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Calculation Sheet		Checked by *RN*	Date *28-09-99*

(vi) Calculation of maximum compressive stress at failure (σ_c):

We have,

$$\sigma_e = \frac{\pi^2 E}{\lambda^2}$$

$$\sigma_c = \phi \pm \sqrt{\phi^2 - f_y \sigma_e} \leq f_y$$

$$\text{where, } \phi = \frac{f_y + (\eta + 1)\,\sigma_e}{2}$$

In x-direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(18.1)^2} = 6025 \ N/mm^2$$

$$\varphi_x = \frac{250 + (0.001 + 1)*6025}{2} = 3140 \ N/mm^2$$

$$\sigma_{cx} = 3140 \pm \sqrt{(3140)^2 - 250*6025} \leq 250$$

$$= 250 \ N/mm^2$$

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	Worked Example - 1		
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In y-direction,

$$\sigma_{ey} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\varphi_y = \frac{250 + (0.141 + 1)585.0}{2} = 459 N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250 * 585} \leq 250$$

$$= 205\ N/mm^2$$

Hence, Allowable axial compressive stress, $\sigma_c = 205\ N/mm^2$

Safe axial compressive stress = $\sigma_c/\gamma_m = 205/1.15 = 178\ N/mm^2$

(vii) Factored Load:

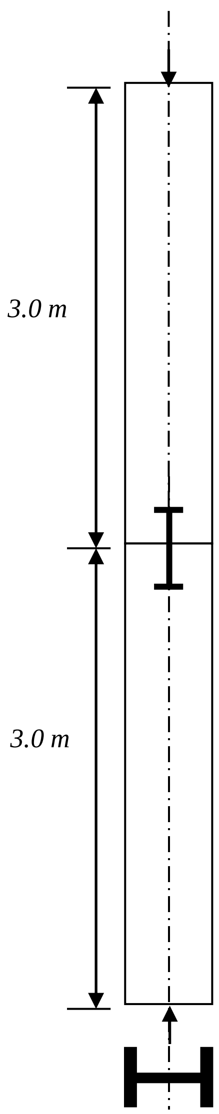
Factored Load = $\sigma_c A/\gamma_m = 178 * 10466/1000$

$= 1863\ kN$

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 2		
Calculation Sheet		Made by *SSSR*	Date 23*-09-99*
		Checked by *RN*	Date 28*-09-99*

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 ; $E = 2.0*10^5$ N/mm^2 ; $\gamma_m = 1.15$]



(i) Type of section:

Section is "COMPACT" from previous example.

(ii) Effective lengths:

$$\ell_x = 6000 \text{ mm}$$
$$\ell_y = 3000 \text{ mm}$$

(iii) Slenderness ratios:

$$\lambda_x = \frac{6000}{166.1} = 36.1$$

$$\lambda_y = \frac{3000}{51.6} = 58.1$$

(iv) Values of (α) and (η):

In x – direction $\alpha_x = 0.002$

In y – direction $\alpha_y = 0.003$

and $\lambda_0 = 17.8$

$$\eta_x = 0.002\,(36.1 - 17.8) = 0.037$$
$$\eta_y = 0.0035\,(58.1 - 17.8) = 0.141$$

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet *2 of 2*	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 2		
		Made by *SSSR*	Date *23-09-99*
		Checked by *RN*	Date *28-09-99*

(v) Calculation of σ_c :

In x- direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(36.1)^2} = 1515\, N/mm^2$$

$$\phi_x = \frac{250 + (0.037+1)*1515}{2} = 911\, N/mm^2$$

$$\sigma_{cx} = 911 \pm \sqrt{(911)^2 - 250*1515} \leq 250$$

$$= 239\ N/mm$$

In y-direction

$$\sigma_{cy} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\phi_y = \frac{250 + (0.141+1)*585}{2} = 459\, N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250*585} \leq 250$$

$$= 205\, N/mm^2$$

(vi) Factored Load:

Factored Load = $\sigma_c A/\gamma_m$ *= 205/1.15* 10466/1000 = 1863 kN*

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Calculation Sheet</td><td>Job No:</td><td>Sheet 1 of 1</td><td>Rev</td></tr>
<tr><td colspan="3">Job Title: BASE PLATE</td></tr>
<tr><td colspan="3">Worked Example - 3</td></tr>
<tr><td rowspan="2"></td><td>Made by SSSR</td><td>Date 20-2-00</td></tr>
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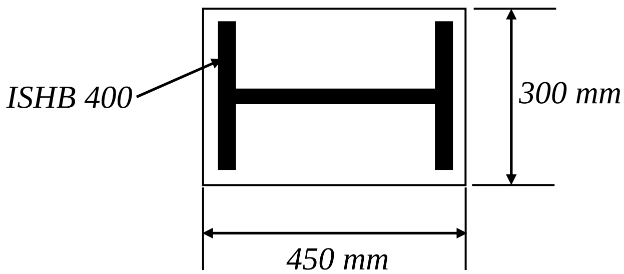
</table>

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 = 1.15]



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

w = (1800 * 10^3) / (450 * 300) = 13.33 N/mm^2

f_{yp} = 250/1.15 = 217.4 N/mm^2

$$t_p = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)}{f_{yp}}} = \sqrt{\frac{2.5\times 13.33\left(25^2 - 0.3\times 25^2\right)}{217.4}} = 8.2\ mm$$

Hence, use 450 X 300 X 10 mm plate.
(Design of connections are discussed in later chapters)