UNIT 6 COMPRESSION MEMBERS

Structure

6.1 INTRODUCTION

Structural members subjected to axial compression are called columns or struts. *Columns* are normally vertical load bearing members in buildings etc., while *struts* may be inclined or horizontal and usually form part of a roof or bridge truss. The axial compression in columns may be accompanied with bending moments, due to external loads or due to eccentricity of loading (load being not exactly axial). Steel columns may be made of single IS rolled section (*simple column*) or a combination of more than one such sections (*compound or built up columns*). Sometimes steel pipes or tubular structures (either hollow or filled up) may be used as columns or struts. Again columns may be *short* or *long*, depending upon the *slenderness ratio*. We have to take special care of failure due to *buckling* in case of columns. All these are explained briefly in this unit with special reference to the IS Code of Practice for General Construction in Steel (IS : 800 – 1984), which has been followed throughout as a sound practice. You are also advised to go through Unit 7 (Columns and Struts) of **Strength of Materials** before reading this unit.

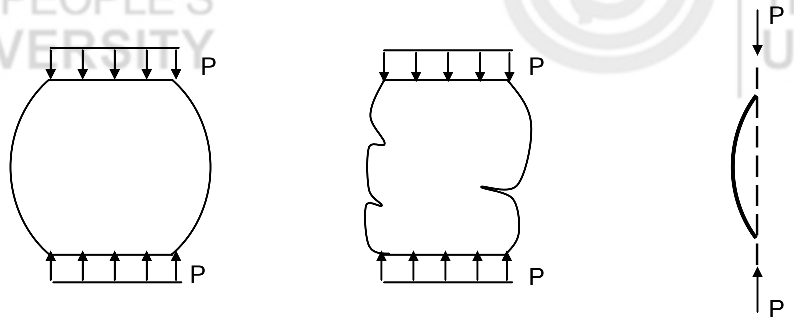
Objectives

After studying this unit, you should be able to

- understand the behaviour of steel structural members subjected to axial compression,
- learn about the various modes of failure of such members especially compression and buckling; and limits of permissible compressive stresses,
- determine the effective length of a column taking into account the support conditions at its ends,
- study the slenderness ratio of columns and its effect in modifying the permissible compressive stresses,
- design the various types of compression members (both simple and built-up) and to check their load carrying capacity, and
- design suitable connections for these compression members.

6.2 COMPRESSIVE LOADING : BUCKLING

The behaviour of a member under compressive load is more complicated than under tensile loads. If a column is short, that is, its lateral dimensions are not very small compared to its length, it may fail by reaching its ultimate compressive strength (yield stress) and the failure may end up in bulging of the section or its cracking up depending upon the material of the column (Figures 6.1(a) and (b)). However, if a column is long, that is its lateral dimensions are small compared to its length, it bends or buckles (deflects from its initial straight position) at a load smaller than that causing pure compressive failure (Figure 6.1(c)).



(a) Failure by Bulging **(b) Brittle Failure** **(c) Buckling**

Figure 6.1

The phenomenon is discussed in the subject of **Strength of Materials** (Unit 7) where it is shown how the 'buckling' may be in three different equilibrium as defined below.

Stable Equilibrium

The element comes back to its original position when the load is removed.

Neutral Equilibrium

It remains in the deflected position as such.

Unstable Equilibrium

The deflection goes on increasing indefinitely causing ultimate collapse.

The ultimate compressive load depends upon :

- **the material of** the column which is expressed in terms of modulus of elasticity (E) and yield stress (σ_y).
- the **effective length** of the column (l_e) which depends upon its end conditions (whether it is pinned, fixed or free).
- its **cross sectional shape and dimensions** which gives its area (A), moment of inertia (I), and radius of gyration $\left(r = \sqrt{\frac{I}{A}} \right)$.

6.2.1 Euler's Theory

One of the earliest mathematical explanation of the phenomenon of buckling was given by Euler (1759) who proposed the following formula (see Strength of Materials, Section 7.2.3).

$$f_E = \frac{P_E}{A} = k\,\frac{\pi^2\,EI}{Al^2} = k\,\frac{\pi^2\,E}{\left(\frac{l}{r}\right)^2} = k\,\frac{\pi^2\,E}{\lambda^2} \qquad \ldots (6.1)$$

where f_E = Euler's critical stress,

P_E = Euler's critical load,

A = Area of cross section,

I = Moment of inertia of the section,

$r = \sqrt{\frac{I}{A}}$ = Radius of gyration,

$\lambda = \frac{l_e}{r}$ = Slenderness ratio of the element, and

k = A coefficient which depends upon the end condition of the column.

Also various empirical formulae, e.g. Rankine's, Johnson's and Perry's are available for estimating the buckling load of columns.

6.2.2 Geometrical Properties of Columns

Area of Cross-section and Member Thickness

According to IS 800 : 1984 the geometrical properties of the gross and effective cross section of a member shall be calculated on the following basis :

(a) The properties of the *gross section* shall be calculated from the specified size of the members;

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

(i) The sectional areas of all holes in the section except that for parts in compression; and

(ii) If the projection of a plate or flange beyond its connection to a web or other support line exceeds the values given in Table 6.1, the excess flange area shall be neglected.

Type	Maximum Allowable Flange Projection
Flanges/plates in compression (with unstiffened edges)	$\dfrac{256\ T_1}{\sqrt{f_y}}$ (maximum 16 T_1)
Flanges/plates in compression (with stiffened edges)	20 T_1 to the innermost face of the stiffening
Flanges/plates in tension	20 T_1

Note :

(i) Stiffened flanges shall include flanges composed of channels or *I*-section or of plates with continuously stiffened edges.

(ii) T_1 denotes the thickness of flange or plate in compression.

(iii) Width of the outstand of members referred above shall be taken as follows :

Type	Width of Outstand
Plates	Distance from free edge to the first row of rivets/welds
Angles, Channels, Z-section and stem of T-section	Nominal width
Flange of beam and T-section	Half the nominal width

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

For Plate in Uniform Compression

$\dfrac{1440\,T_1}{\sqrt{f_y}}$ (subjected to a maximum of 90 T_1).

However, where the width exceeds

$\dfrac{560\,T_1}{\sqrt{f_y}}$ (maximum 35 T_1) for welded (not stress relieved) plates

or $\dfrac{800\,T_1}{\sqrt{f_y}}$ (maximum 50 T_1) for other plates; the excess width shall be assumed to be located centrally and its cross-sectional area shall be neglected.

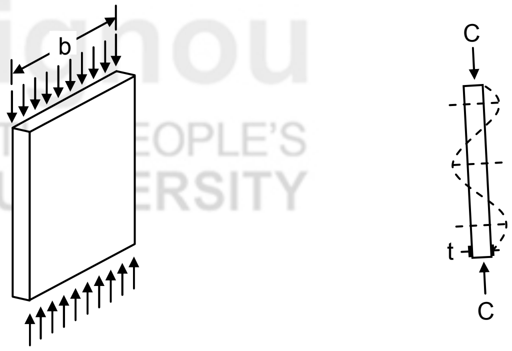
For Plates in Uniform Tension (100 T_1)

However, where the width exceeds 60 T_1 the excess width shall be assumed to be located centrally and its cross-sectional area neglected.

(Here, T_1 is taken to be the web/flange plate thickness of the member.)

Local Buckling

The reason for the above limitations in the effective width of flange or web plates is the phenomenon of *local buckling* which causes wrinkles in the thin plates subjected to compression in their own plane (Figure 6.2).



(a) Thin Plate Under Compression **(b) End View of the Thin Plate under Compression**

Figure 6.2

Due to this phenomenon, the web (or flange) plate will fail before the failure of the actual composite member. As buckling depends upon the *b/t* ratio, the width of these elements are restricted to the values given above. Any width greater than the maximum permissible value is to be ignored in structural calculations. This is explained in Example 6.1.

Example 6.1

Calculate the effective area and effective moment of inertia of the composite column section shown in Figure 6.3 composed of welded flange and web plates. The web plate is 6 mm and the flange plates are 8 mm in thickness. The column has an effective length of 3.4 m (f_y = 250 MPa). Find the slenderness ratio of the column.

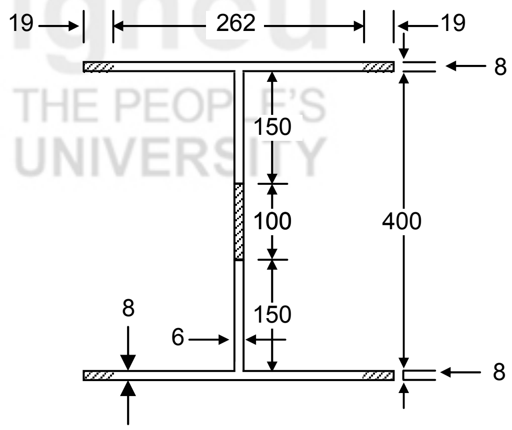


Figure 6.3

Solution

For the flange plate, $\frac{b}{t} = \frac{256}{\sqrt{f_y}} = \frac{256}{\sqrt{250}} = 16.2$ (maximum is 16 t)

Hence, maximum allowable flange projection = 16 × 8 = 128 mm

Actual flange projection $= \frac{300 - 6}{2} = 147$ mm

$\therefore$ Length to be ignored on each end = 147 – 128 = 19 mm

(shown shaded in the Figure 6.3).

For the web plate, $\frac{b}{t} = \frac{1440}{\sqrt{f_y}} = \frac{1440}{\sqrt{250}} = 91.1$

(Maximum allowed is 90 t)

Hence maximum allowed web depth = 90 × 6 = 540 mm

Which is greater than the actual web depth of 400 mm $\therefore$ OK.

However, maximum permissible b/t ratio for the web is

$$\frac{800}{\sqrt{f_y}} = 50.6 \text{ (maximum 50 } t\text{)}$$

Hence, effective depth = 50 × 6 = 300 mm

$\therefore$ The excess depth to be ignored = 400 – 300 = 100 mm and is situated centrally as shown shaded in Figure 6.3.

The effective area of the section is

$$(300 - 19 \times 2) \times 8 \times 2 + (400 - 100) \times 6 = 5992 \text{ mm}^2$$

It must be remembered that all the geometrical properties, e.g. moments of inertia, radii of gyration, slenderness ratio, etc. of the section are modified accordingly.

For example, in the composite section of Example 6.1 above, we have moment of inertia

$$I_{XX} = \left[\frac{262 \times 8^3}{12} + 262 \times 8 \times 204^2\right] \times 2 + \left[\frac{6 \times 400^3}{12} - \frac{6 \times 100^3}{12}\right]$$

$$= 205{,}976{,}629 \text{ mm}^4$$

Similarly, $I_{YY} = \frac{8 \times (262)^3}{12} \times 2 + \frac{300 \times 6^3}{12} = 23{,}985{,}037 \text{ mm}^4$

$$\therefore \quad r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{205976629}{5992}} = 185.4 \text{ mm}$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{23985037}{5992}} = 63.3 \text{ mm}$$

The least radius of gyration = 63.3 mm and the slenderness ratio of the column

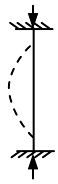
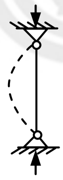
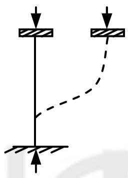
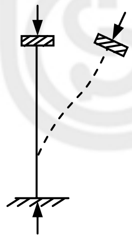
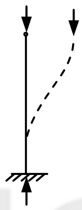
$$\lambda = \frac{l}{r_{\min}} = \frac{3400}{63.3} = 53.7$$

6.2.3 Effective Length of Columns

In the Euler's equation, the column ends are assumed to have pin (or hinged) connections. If the end conditions are different from pins (i.e. fixed, free, etc.) then the formula will not apply as such. To overcome this difficulty, the *actual length* of the column has to be modified by using a multiplying factor.

The length, thus, obtained is called the *'effective length'* of the column. The effective length of columns for various end conditions is given in Table 6.1.

Table 6.1 : Effective Length of Compression Members of Constant Dimensions (Clause 5.2.2)

Case	Degree of End Restraint of Compression Member	Recommended Value of Effective Length	Symbol
(a)	Effectively held in Position and restrained against rotation at both ends.	$0.65L$	
(b)	Effectively held in position at both ends and restrained against rotation at one end.	$0.80L$	
(c)	Effectively held in position at both ends, but not restrained against rotation.	$1.00L$	
(d)	Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position.	$1.20L$	
(e)	Effectively held in position and restrained against rotation at one end, and at other end partially restrained against rotation but not held in position.	$1.50L$	
(f)	Effectively held in position and restrained rotation at one end but not held in position nor restrained against rotation at the other end.	$2.00L$	

Note : (i) L is the unsupported length of compression member.

(ii) For battened struts the effective length shall be increased by 10 percent.

In calculation of the 'slenderness ratio', $\lambda = \frac{l}{r_{min}}$, the value of l is taken as the *'effective length'* of the column.

The maximum slenderness ratio as per recommendation of IS : 800 is 180 for loads resulting from dead and imposed loads. For wind and earthquake forces, this is allowed up to 250, provided the deformation of such members does not adversely affect the stress in any other part of the structure.

The actual strut length, L, shall be taken as the length from centre to centre of intersection with supporting members, or the cantilevered length in the case of free-standing struts.

6.3 PERMISSIBLE STRESSES IN COMPRESSION MEMBERS

As per IS 800 : 1984, the direct stress in compression on the gross-sectional area of axially loaded compression members shall not exceed 0.6 f_y nor the permissible stress σ_{ac}, calculated using the following formula :

$$\sigma_{ac} = 0.6 \frac{f_{cc} \cdot f_y}{\left[(f_{cc})^n + (f_y)^n\right]^{1/n}} \text{ (MPa)} \qquad \ldots (6.2)$$

where f_{cc} = Elastic stress in compression $\text{(MPa)} = \dfrac{\pi^2 E}{\lambda^2}$, [Eq. (6.1)]

f_y = Yield stress of steel (MPa),

E = Modulus of elasticity of steel = 2×10^5 MPa,

$\lambda = \dfrac{l}{r_{\min}}$ = Slenderness ratio of member, and

n = A factor assumed as 1.4.

Values of σ_{ac} for some of Indian Standard structural steels are given in Table 6.2.

It may be observed that for steel having f_y = 250 MPa, the value of allowable σ_{ac} becomes 33 MPa only, for the maximum permissible slenderness ratio of 180, being thus reduced by 87%.

6.4 CROSS-SECTIONAL SHAPES OF COLUMNS

The permissible compressive stress (σ_{ac}) is greatly reduced due to large slenderness ratios (λ) of single member struts. To reduce the value of λ and thus increase the value of σ_{ac}, a compound section consisting of several standard rolled sections is often used.

A few of such arrangements are shown in Table 6.3 whereby the geometrical placement of the sections the value of minimum radius of gyration ($r_{\min}$) is greatly increased, thus, decreasing the slenderness ratio. The followings remarks generally apply.

Rods and square bars, because of their high slenderness ratio, are rarely used as compression members. Single angle (for light loads) and double angles (Tables 6.3(a) to (f)) are the most favoured ones for struts. Single beam sections (e.g. ISHB section) and channel sections are used for larger loads. Groups of four angles are joined together by lacings or rods (Tables 6.3(g) and (h)), giving larger values of the radius of gyration.

Channels and light beam sections provide excellent compound columns with lacing or battens (Tables 6.3(k) to (p)). These are used to carry heavy loads as in multi-storeyed buildings or steel bridge trusses. Riveted or welded plates may also form compound column sections (Tables 6.3(r) and (s)).

Table 6.2 : Permissible Stress σ_{ac} (MPa) in Axial Compression for Steels with Various Yield Stress

(Clause 5.1.1)

$f_y \rightarrow$ / $\lambda \downarrow$	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	132	138	144	150	156	168	180	192	204	215	227	239	251	269	287	305	323
20	131	137	142	148	154	166	177	189	201	212	224	235	246	263	280	297	314
30	128	134	140	145	151	162	172	183	194	204	215	225	236	251	266	280	295
40	124	129	134	139	145	154	164	174	183	192	201	210	218	231	243	255	267
50	118	123	127	132	136	145	153	161	168	176	183	190	197	207	216	225	233
60	111	115	118	122	126	133	139	146	152	158	163	168	173	180	187	193	199
70	102	106	109	112	115	120	125	130	135	139	142	147	150	155	160	164	168
80	93	96	98	101	103	107	111	115	118	121	124	127	129	133	16	139	141
90	85	87	88	90	92	95	98	101	103	105	108	109	111	114	116	118	119
100	76	78	79	80	82	84	86	88	90	922	93	94	96	97	99	100	101
110	68	69	71	72	73	74	76	77	79	80	81	82	83	84	85	86	87
120	61	62	63	64	64	66	67	67	69	70	71	71	72	73	73	74	75
130	55	55	56	57	57	58	59	60	61	61	62	62	63	63	64	64	65
140	49	50	50	51	52	53	53	54	54	54	55	55	56	56	56	56	57
150	44	45	45	45	46	46	46	47	47	48	48	48	49	49	49	50	50
160	40	40	41	41	41	42	42	42	43	43	43	43	43	44	44	44	44
170	36	36	37	37	36	37	38	38	38	38	39	39	39	39	39	39	39
180	33	33	33	33	33	34	34	34	34	35	35	35	35	35	35	35	35
190	30	30	30	30	30	30	31	31	31	31	31	31	32	32	32	32	32
200	27	27	28	28	28	28	28	28	28	28	28	28	28	28	28	28	28
210	25	25	25	25	25	25	26	26	26	26	26	26	26	26	26	26	26
220	23	23	23	23	23	23	23	24	24	24	24	24	24	24	24	24	24
230	21	21	21	21	21	21	22	21	22	22	22	22	22	22	22	22	22
240	20	20	20	20	20	20	20	20	20	36	20	20	20	20	20	20	20
250	18	18	18	18	18	18	18	18	18	19	19	19	19	19	19	19	19

Radii of gyration of some of the common column sections are given in Figure 6.5 and may be used for preliminary design calculations. However, exact calculations must be made for final checking.

Table 6.3 : Compound Columns

6.5 DESIGN OF COMPRESSION MEMBERS

The following points must be noted in the design of compression members :

(a) The thickness of an outstanding leg of any such member must be according to rules laid down in Section 6.2.2.

(b) Except as modified by the above rules the gross sectional area shall be taken for all compression members connected by welds and turned and fitted bolts and pins except that holes which are not fitted with rivets, weld or tight fitting bolts and pins, shall be deducted.

6.5.1 Single Angle Struts

The slenderness ratio (λ) should not exceed 180. Here l_{eff} is taken as the centre to centre of intersections at each end, if connected by a single bolt/rivet. If connected by two or more such rivets/bolts in line of the member or with welds a coefficient of 0.85 may be used (Example 6.2).

The process of *design* (or finding the member size when the load is given) is a trial and error process involving the following steps :

(a) Assuming an arbitrary value of λ, the value of σ_{ac} for the particular σ_y may be taken from Table 6.2.

(b) An approximate sectional area, $A = \dfrac{P}{\sigma_{ac}}$ is calculated and an angle section of this area is selected form IS Handbook No. 1.

The actual effective length (l_{eff}) and the minimum radius of gyration ($r_{\min}$) of the angle chosen is then taken from the Handbook and actual λ is calculated. The corresponding σ_{ac} is compared with the σ_{ac} assumed in Step (a) above. Modifications are made if σ_{ac} (actual) is less than σ_{ac} (assumed) or is much greater than it (difference more than 5%), and a retrial is made with a different section. This is explained in Example 6.3.

Example 6.2

Determine the strength of a sigle angle discontinuous strut, 60 × 40 × 6 if connected by (a) a single rivet, or (b) by two rivets in the line of the member.

Centre to centre between intersections is 1.4 m (f_y of steel is 250 MPa).

Solution

From the tables, Area of 60 × 40 × 6 angle = 5.65 cm^2, r_x = 1.88 cm, r_y = 1.11 cm, $r_{u\,(\max)}$ = 2.01 cm, $r_{v\,(\min)}$ = 0.85 cm.

Case 1 : Single Rivet Connection

$$l_{\text{eff}} = l = 1.4 \text{ m} = 1400 \text{ mm}$$

$$r_{\min} = 0.85 \text{ cm} = 8.5 \text{ mm}$$

Slenderness ratio, $\lambda = \dfrac{\text{l}_{\text{eff}}}{r_{rim}} = \dfrac{1400}{8.5} = 164.1 < 180 \qquad \therefore$ OK.

Elastic critical compression stress, $f_{cc} = \dfrac{\pi^2 E}{\lambda^2}$

Where modulus of elasticity, $E = 2 \times 10^5$ MPa.

$$\therefore \quad f_{cc} = \frac{\pi^2 \times 2 \times 10^5}{(164.7)^2} = 72.8 \text{ MPa}$$

$$\therefore \quad \sigma_{ac} = \frac{0.6\, f_{cc} \cdot f_y}{\left[(f_{cc})^n + (f_y)^n\right]^{1/n}}$$

where $n = 1.4$ Eq. (6.2).

$$\therefore \quad \sigma_{ac} = \frac{0.6 \times 72.8 \times 250}{[(72.8)^{1.4} + (250)^{1.4}]^{1/1.4}} = \frac{72 \times 250}{(404.6 + 2275.7)^{0.714}}$$

$$= \frac{0.6 \times 72.8 \times 250}{281} = 38.9 \text{ MPa}$$

[This may also be obtained by reference to Table 6.2].

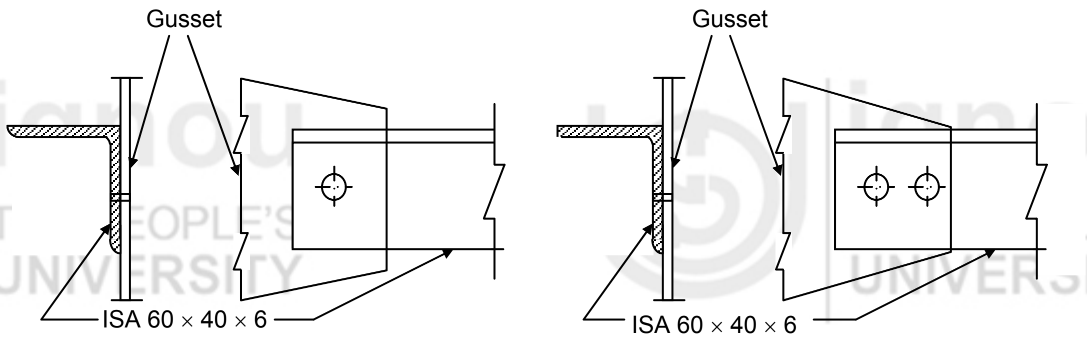
$\therefore$ Actual σ_{ac} allowed is 80% of this value, i.e.

σ_{ac} allowed = 0.8 × 38.9 =31.1 MPa.

Strength of the member = Area × σ_{ac} (allowed)

= 565 × 31.1 = 17571 N ≈ 17.5 kN

This is shown in Figure 6.4(a).



(a) (b)

Figure 6.4

Case 2 : Double Riveted Connection

Here the effective length = 0.85 × Actual length

= 0.85 × 1400 = 1190 mm.

$$\therefore \quad \lambda = \frac{1190}{8.5} = 140$$

By using Eq. (6.2) above or from Table 6.2, you will get the corresponding σ_{ac} as 51 MPa.

σ_{ac} allowed = 0.8 × 51 = 40.8 MPa

and strength of the member = 565 × 40.8 = 23052 N ≈ 23 kN

This is shown in Figure 6.4(b).

Example 6.3

Design a single angle strut of effective length 2.5 m to carry a compressive force of 70 kN. Use steel with f_y = 250 MPa.

Solution

Try an equal angle strut

(a) Choose ISA 80 × 80 × 8,

Area = 12.21 cm^2 , r_{min} = 1.55 cm

$$\therefore \qquad \lambda = \frac{2500}{15.5} = 161$$

Corresponding value of σ_{ac} = 41 MPa (from Table 6.2).

$\therefore$ P allowed = (0.8 × 41) × 1221 N = 40 kN which is too low.

(b) Retry with ISA 100 × 100 × 10, Area = 19.03 cm^2,

r_{min} = 1.94 cm.

$$\lambda = \frac{2500}{19.4} = 129$$

Corresponding value of σ_{ac} = 57 MPa

$\therefore$ P allowed = (0.8 × 57) × 1903 N = 86.7 kN which is too high than the given load of 70 kN.

(c) Hence a third trial is made with ISA 100 × 100 × 8 for which the area is 15.4 cm^2 and r_{min} is 1.94 cm as λ remains the same as in case (b) above. σ_{ac} = 57 MPa and axial load allowed = 0.8 × 57 × 1540 N = 70.2 kN which is ok.

SAQ 1

Design a single angle strut having an effective length of 3 m to carry a compressive load of 100 kN. Take f_y = 250 MPa.

6.5.2 Double Angle Struts

Compression members composed of two angles (channels, or tees), back to back in contact or separated by a gusset, shall be connected together (by rivets or welds) so that slenderness ratio (λ) of each member between connections is not greater than 40 or 0.6 times the most unfavourable slenderness ratio of the strut as a whole (whichever is less). At least two rivets/bolts (or their equivalent welding) should be used to connect them at their ends. Also there shall not be less than two additional connections equidistant in the length of the strut. *Tacking rivets* are provided in double struts, so that they may act together. Their pitch (in line) should be not more than 32 t or 300 mm (whichever is less). Under exposed conditions these values are reduced up to 16 t or 200 mm, where t is the thickness of the outside plate.

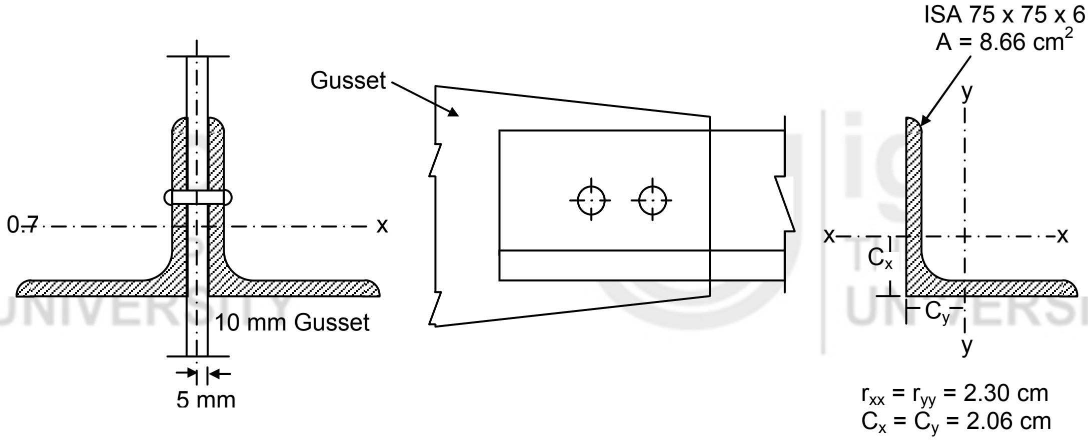
Double angle struts may be connected to the gusset plate, either

(a) back to back to both sides of it by not less than two bolts or rivets (or equivalent welding). The effective length, l_{eff}, is then taken as between 0.7 to 0.85 times the distance between intersections, or

(b) back to back to one side of the gusset by one or more rivets/bolts or equivalent welding in each angle and shall be designed as single angles.

Example 6.4

Design a double angle strut placed back to back but on opposite sides of 10 mm gusset plate to carry a compressive load of 120 kN. The length of the strut between intersections is 2.8 m (f_y = 250 MPa). See Figure 6.5.



(a) **(b)** **(c)**

Figure 6.5

Solution

Choose two ISA 75 × 75 × 6 angles placed as shown in Figure 6.5(a) on opposite side of the gusset plate.

The properties of the angle is shown in Figure 6.5(c).

For double angle, $r'_{xx} = \sqrt{\dfrac{2I_{xx}}{2A}} = \sqrt{\dfrac{I_{xx}}{A}} = r_{xx} = 2.30$ cm , and

$$r'_{yy} = \sqrt{\frac{2\,[(I_{yy} + A(c_y + 5)^2]}{2A}} = \sqrt{\frac{2\,[(Ar_{yy}{}^2 + A(c_y + 5)^2]}{2A}}$$

$$= \sqrt{r_{yy}{}^2 + (c_y + 5)^2} > r_{yy} \text{ or } r_{xx}$$

Hence, $r_{min} = r_{xx}$ = 2.30 cm = 23 mm. Effective length is taken as 0.85 × Intersection distance (as two rows of rivets are provided with suitable tack rivets).

$\therefore \quad l_{\text{eff}} = 0.85 \times 2.8 \text{ m} = 0.85 \times 2800 \text{ mm} = 2380 \text{ mm}$

$\therefore \quad \lambda = \dfrac{2380}{23} = 103$, the corresponding value of σ_{ac} = 77.6 MPa (from Table 6.2), and allowable axial compression

$$= 77.6 \times 2A = 77.6 \times 2 \times 866 \text{ N} = 126.5 \text{ kN}$$

This is about 5% more than the required value, hence may be adopted.

Example 6.5

Design the strut in Example 6.4 if the two angles are placed on the same side of the 10 mm gusset plate.

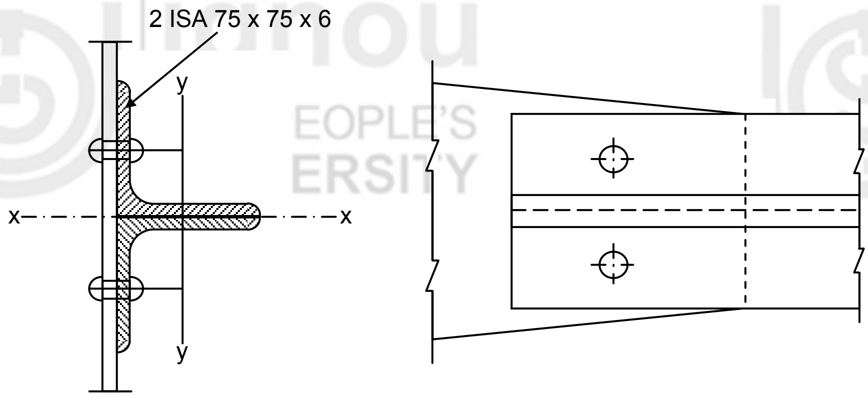


Figure 6.6

Solution

(a) Try two ISA angles 75 × 75 × 8 placed on the same side of the gusset plate as shown in Figure 6.6. The properties of the section are : Area = 11.4 cm^2, $r_{xx} = r_{yy}$ = 2.28 cm, $C_x = C_y$ = 2.14 cm.

The radius of gyration about axis xx

$$r'_{xx} = \sqrt{\frac{2\,(I_{xx} + AC_x^2)}{2A}} = \sqrt{r_{xx}^2 + C_x^2} > r_{xx}$$

The radius of gyration about axis yy

$$r'_{yy} = \sqrt{\frac{2I_{yy}}{2A}} = r_{yy} \, .$$

As $r_{xx} = r_{yy}$ for the equal angles, the second case will give smaller value of radius of gyration (about y-y axis).

$\therefore$ r_{xx} of the double angle section = 2.28 cm = 22.8 mm.

$\therefore$ As each angle is connected to the gusset plate by one rivet only the effective length is equal to actual length, i.e. l_{eff} = 2800 mm.

$\therefore$ Slenderness ratio, $\lambda = \dfrac{2800}{22.8} = 123$, for which corresponding σ_{ac} = 62 MPa.

Here the allowable stress will be 80% of σ_{ac}.

$\therefore$ $P_{\text{allowable}}$ = (0.8 × 62) × (2 × 1140) N =113 kN < 120kN $\therefore$ rejected.

(b) Next try two ISA 80 × 80 × 8 sections similarly placed

Area = 2 × 12.2 cm^2 = 2440 mm^2, $r_{xx} = r_{yy}$ = 2.44 cm

$\therefore$ $\lambda = \dfrac{2800}{24.4} = 115$, for which corresponding σ_{ac} = 68 MPa.

$\therefore$ $P_{\text{allowable}}$ = 0.8 × 68 × 2440 N = 132.7 kN.

As this is larger than the required 120 kN load, we have to accept it, as the standard section (IS angles) of next lower size (i.e. 2 nos ISA 75 × 75 × 8) was found to be insufficient.

SAQ 2

Design a 'Double Angle Strut' placed back to back to carry a compressive load of 100 kN. Effective length of the strut is 3 m (f_y = 320 MPa).

6.5.3 Built-up Sections

Built-up sections are used as compression members for heavy loads and consist of one or more solid rolled steel section with flanges having cover plates. The cover plates are connected to the flanges by rivets or welds. These connections are designed to resist a shear force, which is 2.5% of the maximum compressive load on the column.

Example 6.6

Figure 6.7 shows an ISHB 350 @ 72.4 kg/m with two flange plates 400 × 10 mm section each. Find the load carrying capacity of the column if the effective length is 4.5 m (f_y = 250 MPa).

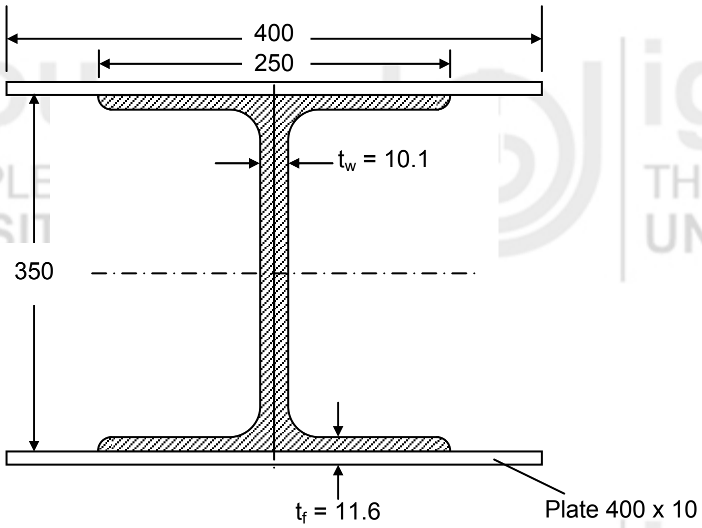


Figure 6.7

Solution

Area of the ISHB section = 92.21 cm^2

$$I_{xx} = 19802.8 \text{ cm}^4$$

$$I_{yy} = 2510.5 \text{ cm}^4$$

The minimum radius of gyration will be about the minor *y*-*y* axis.

$$I_{yy} \text{ of compound section} = 25{,}105{,}000 + 2 \times \frac{10 \times 400^3}{12}$$

$$= 131,\ 771,\ 667 \text{ mm}^4$$

Area of compound section = 9221 + 2 × 400 × 10 = 17221 mm^2

$$\therefore \qquad r_{\min} = \sqrt{\frac{131,771,667}{17,221}} = 87.5 \text{ mm}$$

$$\therefore \qquad \lambda = \frac{4500}{87.5} = 51.4$$

Corresponding allowable σ_{ac} = 130.6 MPa (from Table 6.2)

$\therefore$ Load carrying capacity = σ_{ac} . A = 130.6 × 17221 N = 2249 kN.

SAQ 3

Design a built-up column section (using ISHB sections with suitable flange plates) to carry an axial load of 2500 kN. The effective length of the column is 5.5 m.

6.5.4 Tubular (Pipe) Columns

Steel or cast iron tubes and pipes have a larger radius of gyration ($r = 0.35\ d_m$ where d_m is the mean diameter of the pipe) than solid steel rods of the same cross sectional area. Hence, the slenderness ratio is lowered and the corresponding permissible stress in compression is increased. Standard steel tubes of various nominal bores are available. They are normally available in three classes : *light, medium* and *heavy*. The sectional properties are tabulated in IS Handbook-1, which may be referred.

Example 6.7

Find the load carrying capacity of a steel tubular column of 3 m length. One end of the pipe is fixed and the other end is pinned to the supports. The nominal bore of the heavy class pipe is 100 mm.

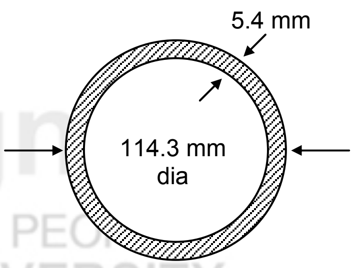


Figure 6.8

Solution

Properties of the 100 bore heavy pipe :

Outside diameter = 114.3 mm; thickness = 5.4 mm

Area of cross section = 18.5 cm^2, radius of gyration = 3.85 cm = 38.5 mm.

The effective length of the column from Table 6.1 case (b) is
0.80 × 3 = 2.4 m = 2400 mm.

$\therefore$ Slenderness ratio, $\lambda = \dfrac{2400}{38.5} = 62.3$

The corresponding value of σ_{ac} (for $f_y = 250$) is 119.7 MPa (Table 6.2).

$\therefore$ Load carrying capacity = $\sigma_{ac} .A = 119.7 \times 1850$ N = 221.4 kN.

SAQ 4

A steel tubular column of 3.5 m length is pinned at one end and fixed at another. Design a suitable heavy class pipe section to carry an axial load of 250 kN.

6.5.5 Encased Columns

Sometimes structural sections are encased in masonry or concrete. This improves its slenderness ratio and thus enhances its strength. It also increases its durability and attack from harmful chemicals or fumes. However the IS : 800 stipulates some conditions to be fulfilled by them :

(a) The steel section must be symmetrical (single I or double channel back to back).

(b) The overall dimension of the steel section must not be greater than 750 × 450 mm (750 parallel to web).

(c) The column should be unpainted and solidly encased in ordinary dense concrete of minimum M-15 grade with 20 mm aggregates.

(d) The minimum width of solid casing to be (b_0 + 100) where b_0 is the width of steel flange (mm).

(e) The surface and edges of the column should have a concrete cover of not less than 50 mm.

(f) The casing is effectively reinforced with steel stirrups or binders (minimum 5 mm dia) at a minimum pitch of 150 mm, so arranged as to pass through the centre of the covering of the edges and outer faces of the flanges and supported by at least 4 longitudinal spacing bars.

Design Considerations

(a) The steel section shall be considered as carrying the entire load, but allowances may be made by assuming the radius of gyration *'r'* of the column section about the axis in the plane of its web to be 0.2 (b_0 + 100). The radius of gyration about its other axis shall be taken as that of the uncased section.

(b) The axial load on the encased column < 2 × (axial load permitted on uncased section).

(c) Slenderness ratio of uncased section for its full length centre to centre of connection) < 250.

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

(e) The allowable compressive load P shall be determined as follows :

$$P = A_{sc}\,\sigma_{sc} + A_c\,\sigma_c , \qquad \ldots (6.3)$$

where, A_{sc} , A_c = Cross sectional area of steel and concrete; and

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

Example 6.8

Determine the load carrying capacity of an ISMB-500 section encased in concrete with 50 mm cover all round. The length of the column is 3 m and it is effectively held in position and restrained against rotation at one end, and at the other end restricted against rotation but not held in position (f_y = 250 MPa and M 15 of concrete) is used.

Solution

Effective length of the column (case (d) of Table 6.1) is given by

$$l_{\text{eff}} = 1.20\text{ L} = 1.20 \times 3\text{ m} = 3.6\text{ m}$$

Properties of the Steel section are : Sectional area (A) = 110.74 cm^2

r_{xx} = 20.21 cm; r_{yy} = 3.52 cm; Flange width b_0 = 180 mm

Slenderness ratio for uncased column about y-y axis

$$= \frac{3600}{35.2} = 102.3 \ (< 250 \ \therefore \ \text{OK})$$

Slenderness ratio for encased column $= \dfrac{3600}{0.2\,(180 + 100)} = 64.3$

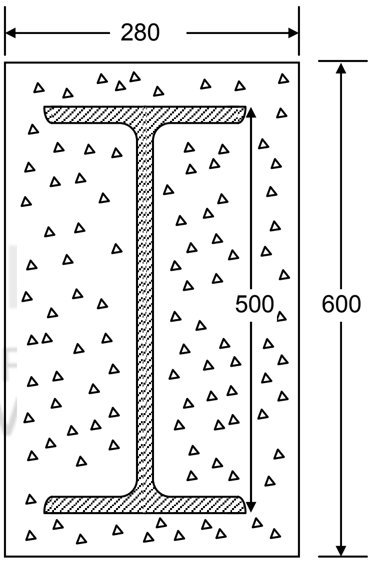


Figure 6.9

$\therefore$ σ_{ac} for uncased column = 78.2 MPa

σ_{ac} for cased column = 117.7 MPa

σ_c for concrete Mix M 15 = 4 MPa

Area of concrete = 280 × 600 – 11074 mm^2 = 156926 mm^2

∴ Land carrying capacity of cased column = 117.7 × 11074 + 4 × 156925

= 1931114 N ≈ 1931 kN.

Load carrying capacity of uncased column = 11074 × 78.2 = 865987 N

≈ 866 kN.

The carrying capacity of encased column cannot be greater than

2 × 866 = 1732 kN by Design Considerations (b) above, hence the value of 1732 kN (instead of 1931) is to be taken.

6.6 DESIGN OF COMPOUND COMPRESSION MEMBERS

You have observed that the load carrying capacity of compression members is greatly influenced by its slenderness ratio, a larger value of which reduces the allowable compressive stress. As the effective length is mostly fixed in such members, it is only the radius of gyration, which can be manipulated by the designer. In most of the rolled steel sections the minor radius of gyration is much smaller than the major one, thus reducing the strength of the member. This can be overcome by arranging two or more sections such that the radii of gyration about the two principal axes are approximately the same. A number of such arrangements are shown in Figure 6.10.

Here two channels (Figures 6.10(a) and (b)) or two I-sections (Figure 6.10(c)) or four angle section (Figures 6.10(d), (e) and (f)) are joined together by suitable connecting system (shown by dotted lines in the figure). An exact spacing between the sections can be obtained by equating the moments of inertia $I_{xx} = I_{yy}$ of the compound member. However, an approximate value for the double channels is given in Figures 6.10(a) and (b).

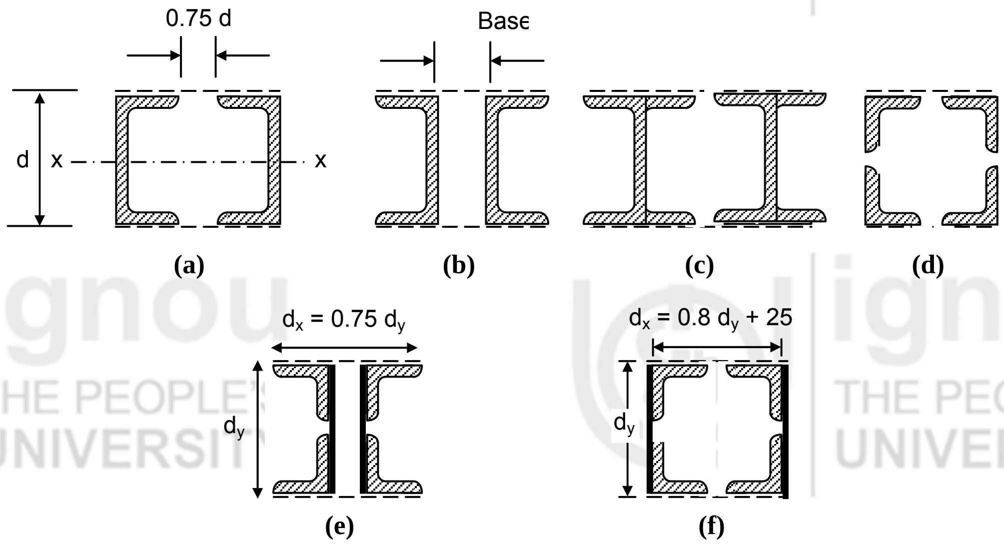


Figure 6.10

For the four angles joined with plates (Figure 6.10(e)) if $d_x \approx 0.75\ d_y$, then I_y will be not less than I_x. For the arrangement in Figure 6.10(f), keep $d_x \approx 0.8\ d_y + 25$ mm.

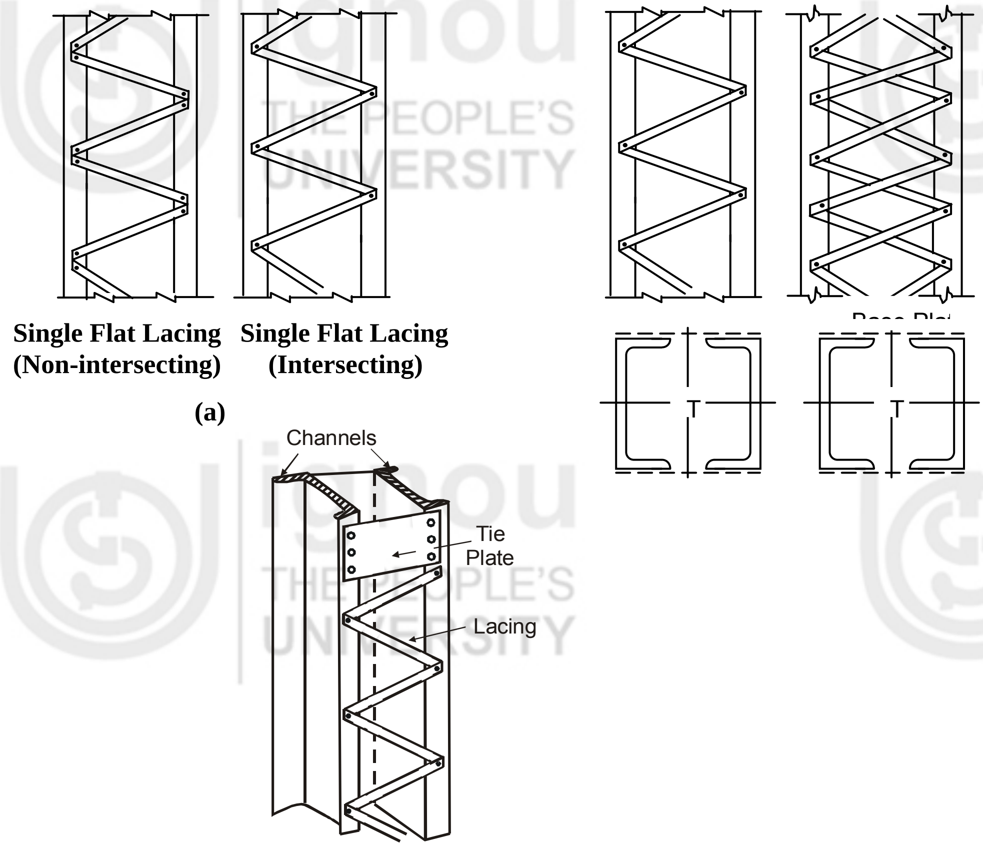
The main structural elements along with the connecting system must act as one element, without any joint failure, relative movement or distortion.

As regards the lateral connecting system, shown by dotted lines in Figure 6.10, is concerned, two common methods used in structural engineering practice are (a) Lacing, and (b) Battened.

Depending upon the system adopted we have different design procedures for each one of them.

6.6.1 Compound Columns with Lacing

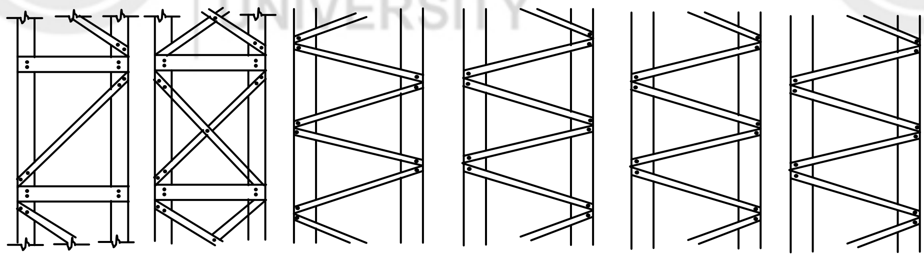
Lacings normally consist of 50-65 mm wide mild steel flats or strips. But rolled steel sections (angles, rods or tubes) of equivalent strength may also be used. These are generally inclined in direction. Lacings may be intersecting or non-intersecting type, depending whether their ends overlap or not (Figure 6.11(a)). Also lacing may be single-laced or double-laced as in Figure 6.11(b).



(c) Isometric View Laced Column (Back to Back Channel)

Figure 6.11

As far as practicable the lacing system shall not be varied throughout the length of the strut. IS : 800 does not recommend use of cross members (perpendicular to the longitudinal axis of the strut) along with single- or double-laced system of connections (Figure 6.12) except for the tie plates or end tie plates.



Laced Systems with Cross Members (Not Recommended) **Face A** **Face B** **(a) Preferred System** **Face A** **Face B** **(b) Not Preferred**

Figure 6.12

Single-laced systems on opposite sides of the components shall preferably be in the same direction so that one be the shadow of the other (Figure 6.12(a)) instead of being initially opposite in direction (Figure 6.12(b)).

Design Considerations

The load carrying function is performed by the *main structural members*. The lacing bars (or battens) are not load carrying elements. Their chief function is to hold the main component members in position and equalize the stress distribution among them. However, they are expected to resist some lateral or eccentric loading which tend to distort the system, and cause buckling of the individual structural elements separately. The following conditions are to be considered :

(a) Buckling of the column as a whole.

(b) Buckling of a component of the column (between connections).

(c) Failure of the latticed lacing members due to a horizontal shear.

(d) Distortion of the cross-section of the built-up column due to bending moment or shear force.

The IS 800 : 1984 has given the following recommendations regarding design of lacings :

(a) The lacings are designed to resist a total *transverse shear*, *V*, at any point in the length of the member, equal to at least 2.5% of the axial force in the member. This shear is considered as divided equally among all transverse lacing systems in parallel planes.

Thus, $$V = \frac{P}{40} \qquad \ldots (6.4)$$

(b) If any *additional shear* is caused due to eccentricity of loading, applied end moments or lateral loading on the column, this has to be added to *V* in Eq. (6.4).

(a) The *slenderness ratio* of the lacing bars shall not exceed 145. The effective length of the lacing bars are calculated as follows :

(i) For single lacing with riveted connection, it is the length between the inner end rivets.

(ii) For double lacings (effectively riveted at their intersections), it is 0.7 times this length.

(iii) For welded construction, it is 0.7 times the distance between the inner ends of the connecting welds.

(d) *Width of lacing bars* depends upon the nominal rivet diameter as given in Table 6.4 below.

Table 6.4 : Width of Lacing Bars

Nominal Rivet Dia (mm)	Width of Lacing Bar (mm)
22	65
20	60
18	55

16	50

(e) *Thickness of* lacing bars shall not be less than the following :

(i) $\frac{l}{40}$ for single riveted lacing;

(ii) $\frac{l}{60}$ for double riveted lacing;

(iii) $\frac{l}{60}$ for welded lacing;

where l is the length between inner end rivet/welds.

(f) *Angle of Inclinations* : Whether in single system or double system the lacing bars shall be inclined neither less than 40^o nor more 70^o to the axis of the member.

(g) *Spacing* : The maximum spacing of the lacing bars (whether riveted or welded) shall be such that the minimum slenderness ratio, $\lambda' = \frac{l'}{r'}$ of the components of the column between connections is not greater than either 50 or 0.7 λ (where λ is the most unfavourable slenderness ratio of the member as a whole), whichever is less.

(h) *Attachment to Main Members* : The riveted or welded connection of the lacing bars to the main members shall have sufficient strength so as to transmit the load in the bars. Where welded lacing bars overlap the members, the amount of lap measured along either edge of the lacing bars shall not be less than 4t (where t is the thickness of the bar or member, whichever is less). Welding shall be provided along each side of the bar for the full length of the lap. Welds are to be fillet welds or full penetration butt welds.

Steps for Design

The following steps are to be taken while designing compound compression members with lacing :

(a) Select a suitable arrangement of structural members (e.g. one as shown in Figure 6.10). The size of the open columns usually lies between $\frac{1}{9}$ to $\frac{1}{14}$ of the column length.

(b) Assuming a reasonable value of λ, find the corresponding σ_{ac} from Table 6.2 and find the total area of the column $A = \frac{P}{\sigma_{ac}}$.

From your assumed value of λ, also find $r_{\min} = \frac{l_{\text{eff}}}{\lambda}$.

(c) From the total area A, find the individual area of each element and select suitable structural sections from the IS : Handbook-1. From the tabulated values of r'_{xx} and r'_{yy} and A', of each component find r_{xx} and r_{yy} of the compound column, so that both are approximately equal. In this, help of Table 6.3 may be taken.

(d) Finally, for the selected structural sections, calculate the exact A, $r_{\min}$, l_{eff}, $\lambda_{\max}$ etc. From the last data find the corresponding

σ_{ac} from Table 6.2 and check up with your assumptions in step (b). If there is a discrepancy repeat the process.

(e) Select the angle of inclination of the lacing bars ($40^o < \theta < 70^o$; normally 45^o), their maximum spacing, width, thickness, etc. (If the single-lacing system gives a large value of l' such that $\theta > 70^o$, then adopt double-lacing system.)

(f) Design the lacing for a transverse load of $V = \frac{P}{40}$. Divide total V between the number (N) of lacing systems provided. The forces (compressive or tensile) in the lacing bars will be $\frac{V}{N}$ cosec θ as shown in Figure 6.13.

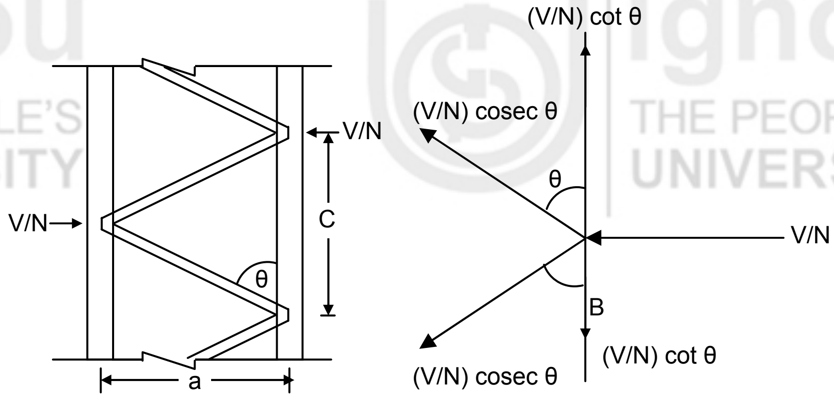


Figure 6.13 : Forces Acting on Lacings and Tie Plate

(g) *End tie plates* are provided at end of the column and are designed as battens (see next section)

Bending moment in tie plate = $\frac{VC}{2\,N}$, and

$$\text{Shear force} = \frac{VC}{a\,N}$$

where, C = Spacing of lacings, and

a = Centre to centre length of tie plate.

This is illustrated in Example 6.9.

Example 6.9

Design a built-up column using single lacings to carry an axial load of 1200 kN. The connections are all welded (σ_y = 250 MPa). The effective length of the column is 5 m.

Solution

Select two ISMC-300 channels, back to back as shown in Figure 6.15. The properties of the section are :

Area = 45.64 cm^2 = 4564 mm^2; r_x = 11.81 cm = 118.1 mm

r_y = 2.61 cm = 26.1 mm; c_y = 2.36 cm = 23.6 mm

We know that r_x for the double system is same as that for the single channel

$\therefore$ r_x = 118.1 mm

and $\quad r_y = \sqrt{r_y'^2 + \left(c_y + \frac{d}{2}\right)^2}$

where r_y' is the radius of gyration of the single angle; substituting the values

$r_y = \sqrt{26.1^2 + (23.6+90)^2} = 116.5$ mm which is approximately equal to r_x ($r_x = 118.1$).

Since r_y is the smaller radius of gyration.

$\therefore$ Slenderness ratio, $\lambda = \frac{5000}{116.5} = 42.9$.

Corresponding value of $\sigma_{ac} = 137$ MPa.

$\therefore$ $P_{\text{allowable}} = 2 \times 137 \times 4564$ N $= 1250$ kN > 1200 $\quad\therefore$ OK.

Design of Lacing

Check for Spacing

From Figure 6.14(b) distance between CG of channels

$= 180 + 2\, c_y = 180 + 2 \times 23.6 = 230$ mm

Using angle of lacings = 45°, distance (vertical) between connections

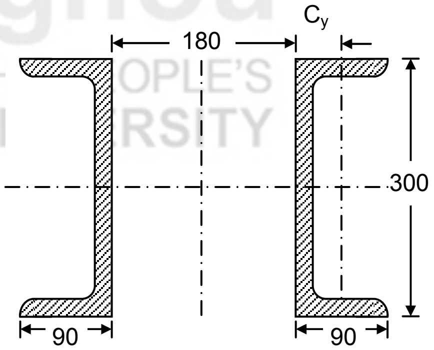
$= 2 \times 230 = 460$ mm.

$r_{\min}$ of the single (channel) element $= r_y = 26.1$ mm.

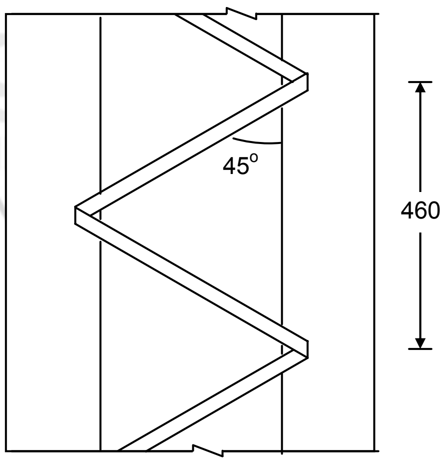
$\therefore$ $\lambda' = \frac{l'}{r'} = \frac{460}{26.1} = 17.6$; This is less than either

(i) 50 or

(ii) $0.7 \times \lambda = 0.7 \times 42.9 = 30$ $\therefore$ OK.



(a)



(b)

Figure 6.14

Design for Horizontal Shear

Horizontal shear, $V = \frac{P}{40} = \frac{1200}{40} = 30\,\text{kN}$

$\therefore$ Force in lacing, $F = \pm\left(\frac{V}{N}\right)\text{cosec}\,\theta = \frac{30}{2}\,\text{cosec}\,45^o$

$= 21.213$ kN

$\therefore$ Length of lacing, $l = 230\sqrt{2} = 325\,\text{mm}$

t_{min} for lacing $= \frac{325}{40} = 8.1$ mm, adopt say 10 mm thickness.

Try 60 × 10 mm flat MS section for lacings.

$$r_{\min} = \frac{t}{\sqrt{12}} = \frac{10}{\sqrt{12}} = 2.9\text{ mm} \quad \therefore \lambda = \frac{325}{2.9} = 112$$

Corresponding value of σ_{ac} = 70.4 MPa (from Table 6.2)

$\therefore$ $F_{\text{comp}} = 70.4 \times 600 = 42{,}240\text{ N} > 21213\text{ N}$ $\therefore$ OK.

Allowable tensile stress in lacing, $\sigma_{at} = 0.6 \times f_y = 0.6 \times 250$

$= 150$ MPa

$\therefore$ $f_{\text{ten}} = 150 \times 600 = 90000 > 21213\text{ N}$ $\therefore$ OK.

Design of Welds

Using 4 mm fillet welds,

Weld strength, $R = 108 \times (0.7 \times 4) = 304$ N per mm length

$\therefore$ Length of weld $= \frac{F}{2R} = \frac{21213}{2 \times 304} = 35\text{ mm}$.

Provide weld length = 40 mm.

Overlap provided > 4 × thickness = 4 × 8 = 32 $\therefore$ OK.

SAQ 5

Design a built-up steel column using single lacing system to carry an axial load of 2000 kN (f_y = 250 MPa). The effective length of the column is 6 m.

6.6.2 Compound Columns with Battens

In this system, the connections between the main component members are made by horizontal connecting pieces or battens, which may be of plates, angles, channel, or I-sections riveted or welded as the case may be (Figure 6.15).

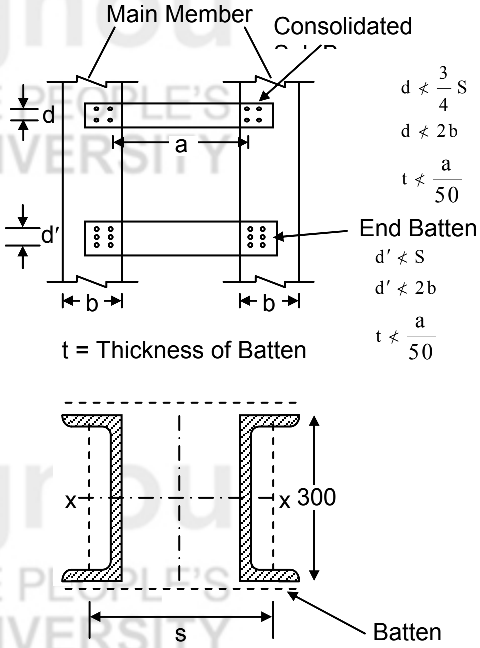


Figure 6.15

6.7 FOUNDATIONS FOR STEEL COLUMNS

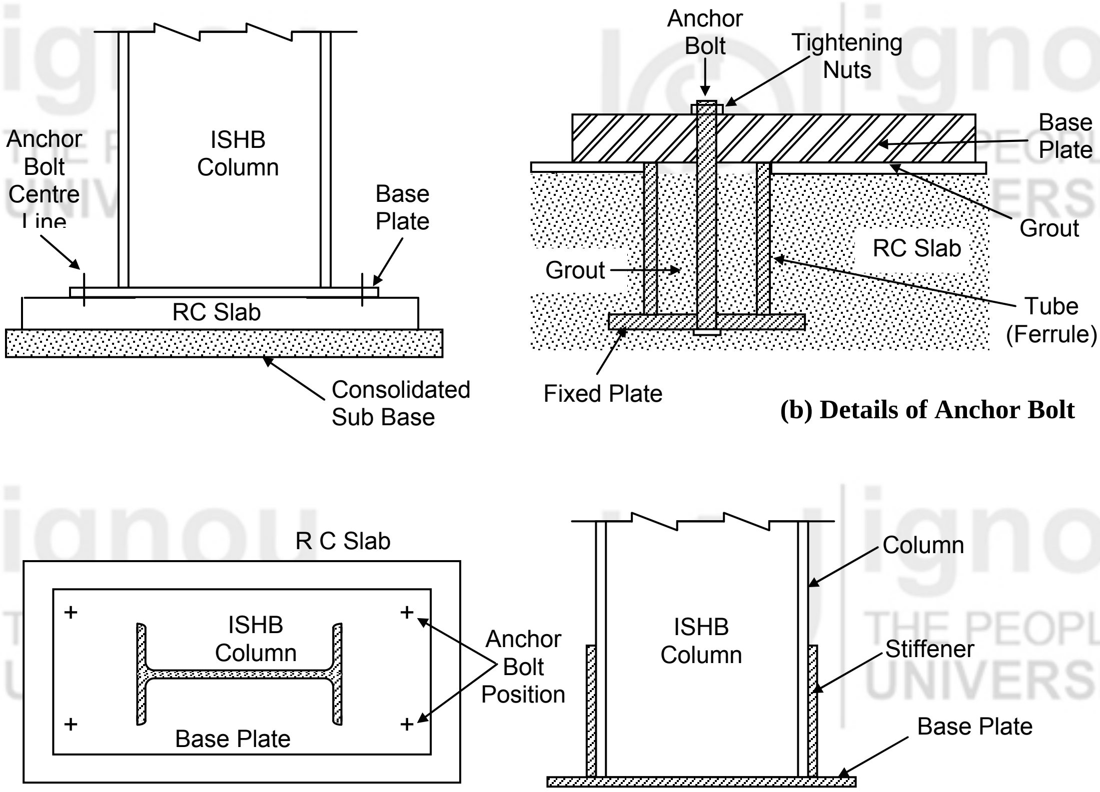
The loads carried by steel columns are transmitted to the soil through suitable devices which are known as *column bases*. In fact, the bearing capacity of the soil being much smaller than the strength of steel, the area of the foundation required is much larger than the area of the column to carry the same load. Also steel is liable to suffer corrosion etc. in contact with soil, hence special care has to be taken to connect the column to the actual foundation to ensure :

- proper alignment and verticality of columns;
- protection of steel base from ground corrosion and damage; and
- proper and firm anchorage to the column base, limiting any foundation settlement or movement.

Some of the common type of column foundations are described below.

6.7.1 Slab Base

Figure 6.16(a) shows a slab base, which consists of a *reinforced concrete slab*, which is laid at a suitable depth after providing a firm *sub-base* or screed. The load from the column to the RC slab is transmitted through a flat horizontal steel *base plate*. The steel column is either riveted or welded to the base plate, through a proper connection. The plate is fixed to the RC slab by means of suitable *hold-fast* (*or anchor*) *bolts*, which is then properly grouted with cement. In order to allow gradual increase in area an RC pedestal is often provided over the RC slab. The anchor bolts are tightened such that they develop a tensile stress, which is approximately 90% of the proof stress. They are enclosed in a ferrule or pipe, which is embedded in the concrete as shown in Figure 6.16(b). A tolerance of about 2 mm is provided in the bolt holes for minor alignment of the column. The alignment of columns and their levels are properly checked before final grouting is done.



(b) Details of Anchor Bolt

(a) Slab Base for Steel Columns **(c) Stiffened Slab Base**

Figure 6.16 : Slab Base

The base plates are designed to withstand the axial load, horizontal shear or external moments transmitted through the base of the column, and should be of sufficient area to avoid crushing of concrete. The concrete RC slab in its turn should be of sufficient area to distribute the load to the sub-base such that the upward soil pressure does not exceed the safe bearing capacity of the soil. The RC slab is designed to withstand stress due to this upward soil pressure. For larger loads, sometimes *stiffeners* are needed for the base plates (Figure 6.16(c)).

6.7.2 Grillage Base

If the column loads are very heavy, or the bearing capacity of the soil is very low, the foundation area required becomes large; too large for providing a suitable base plate and RC pedestal type foundation. One of the methods to disperse the column load to such a large area is by providing several tiers (at least two) of steel grillage beams.

The grillage beams are encased in lean concrete, so as to provide safety against corrosion and deterioration due to contact with soil. Figure 6.17 shows a two-tier grillage base. The upper tier beams are connected to the column through a base plate, which may be gusseted and stiffened as per load requirements. The lower tier beams on which the upper tier rests finally transmit the load to the soil. The IS : 800 allows an increase of $33\frac{1}{3}\%$ in the usual permissible stresses (50% in case of wind/earthquake stresses included) in the design of grillage beams. The following conditions, however, are to be fulfilled :

(a) Beams are not to be painted and should be solidly encased in (minimum) M-15 concrete with 10 mm size aggregates.

(b) Adjacent beam flanges to have clear spacing of 75 mm minimum.

(c) Side and top concrete cover to the beams should be at least 100 mm all round the edges/flanges of beams.

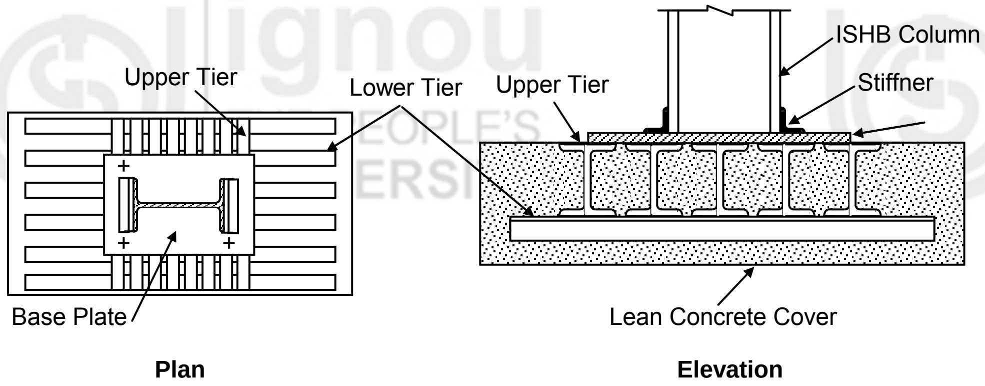
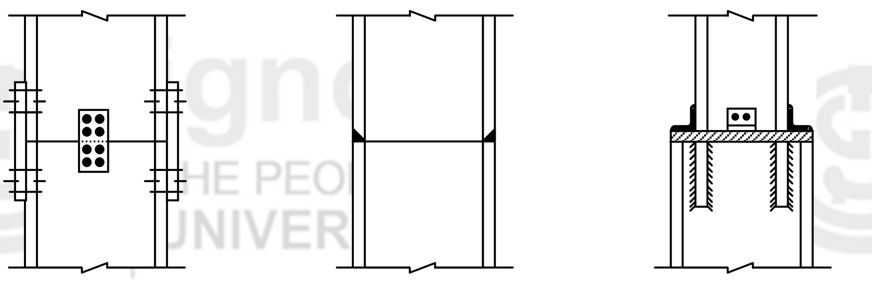


Figure 6.17 : Grillage Base (Details)

6.7.3 Column Splice

The length of structural steel members as available in the market or site is limited up to 15 m maximum, or even less. In multistoreyed buildings, for convenience of fabrication it is kept at about 5 m lengths. Also the cross sectional area of columns may vary from one storey to another. Due to these reasons columns are joined with each other along their lengths. Such joints are called *column splices.* The design of the splice depends upon the surface quality of the column ends at the joint. If the ends are machined and milled, so as to provide good contact area along the whole cross-section, the splice plates are free from any force transmitted. However, they have to be designed against tensile forces caused by lateral (wind/earthquake) loading and also due to erection stresses. If the ends of the columns are not milled, the forces from the upper column is transmitted to the lower one through the splices. This is to be designed to transmit all the forces to which they are subjected as a single cover butt joint welded or riveted as the case may be. Various types of column splices are shown in Figure 6.18.

IS : 800 stipulates that column splices and butt joints of struts and compression members depending on contact for stress transmission shall be accurately machined and close butted over the whole section with a clearance not exceeding 0.2 mm locally at any place. The ends of shafts together with the attached gussets, angles, channels, etc. after riveting together should be accurately machined so that the parts connected butt over the entire surfaces of contact. (Reduction in thickness by machining should not be more than 2 mm.)



(a) Column Splice Rivetted **(b) Column Splice Welded** **(c) Column Splice with Different Section**

Figure 6.18 : Column Splices

6.8 SUMMARY

Compression members in steel need special care in design. Steel possesses high strength in tension/compression, hence the sections are generally small compared to the length, resulting in slender elements. These elements, as you have seen after studying Euler's buckling theory, are easily subjected to failure due to buckling. Thus, the strength of a compression member is strongly affected by its slenderness ratio, which must be estimated before hand. You have studied the various methods of failure of columns and struts. Various cross-sections' shapes are available for compression members and the best shape is to be selected depending upon the external loading and the location of the member. Designs of pipes or tubular elements as columns as well as encased columns have been discussed. The designs of both axially loaded columns as well as columns subjected to eccentric loading have been explained.

6.9 ANSWERS TO SAQs

SAQ 1

Try $ISA\ 110 \times 110 \times 10$ Angle section

Area = 21.06 cm^2, r_{rmin} = 2.14 cm

$$\lambda = \frac{l}{r} = \frac{300}{2.14} = 140 \ \rightarrow\ \text{corresponding } \sigma_{ac} = 51 \text{ MPa (from tables)}$$

Allowable load $= 2106 \times 51 = 107406\ \text{N} > 100000\ \text{N}$ $\therefore$ OK .

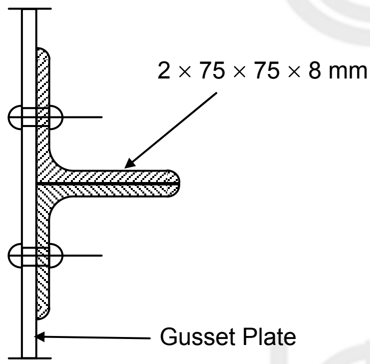
SAQ 2

Angles placed on the same side of gusset plate

Try $ISA\ 75 \times 75 \times 8$ angle

$$A = 11.38\ \text{cm}^2 \qquad r_{xx} = 2.28$$

r_{min} of double angle = 2.28 cm



$$\therefore \qquad \lambda = \frac{l}{r} = \frac{300}{2.28} = 132 \ \rightarrow\ \text{corresponding } \sigma_{ac} = 58.8 \text{ MPa for } f_y = 320.$$

$\therefore$ Allowable $\sigma_{ac} = 0.8 \times 58.8 = 47.04$ MPa

Maximum axial load, $P = 2 \times 1138 \times 47.04 = 107063\ \text{N} > 100000\ \text{N} \quad \therefore$ OK.