6 Compression Members

6.1 : INTRODUCTION

Pure compression members are structural elements subjected to axial compressive forces only. Axial compressive force means forces applied along the centroid of a longitudinal axis of the cross section. A column is defined as a structural member whose longitudinal dimension is comparatively more than its lateral dimensions and is predominantly subjected to compressive force in a direction parallel to its longitudinal axis. End posts are the end compression members in truss bridge girders. The structural members carrying compressive load in a truss are called struts. Heavy compression members in building are called stanchions. The compression member of a crane is called a boom. The compression members may also be subjected to both axial compression and bending. In this chapter, the compression members subjected to axial loads only are discussed.

6.2 : COMMON SHAPES OF COMPRESSION MEMBERS

Rolled steel sections are generally used as compression members. Different types of sections and their uses are described in Table 6.1.

Table – 6.1.

Sl No.	Types of rolled steel sections	Uses
1.	Single angles	1. Light roof trusses, bracing in plate girders and built up columns
2.	Double angles back to back	2. Top chord members of roof truss
3.	T sections	3. Welded roof truss
4.	Single channels	4. Rarely used as columns. Not suitable due to low value of radius of gyration.
5.	Circular hollow sections	5. Tall buildings. Most efficient due to equal value of radius of gyration in every axis.
6.	Square and rectangular hollow sections	6. Tall buildings.Easy fabrication and erection
7.	I sections	7. Suitable for columns. Difference in radius of gyration about two axes is the smallest.
8.	Built up sections	8. Compression members subjected to heavy loads.

The choice of a particular section depends on its availability in the market, problem to connect with other stuctural components and slenderness ratio. The different cross sections shapes for rolled steel compression members are shown in figure 6.1.

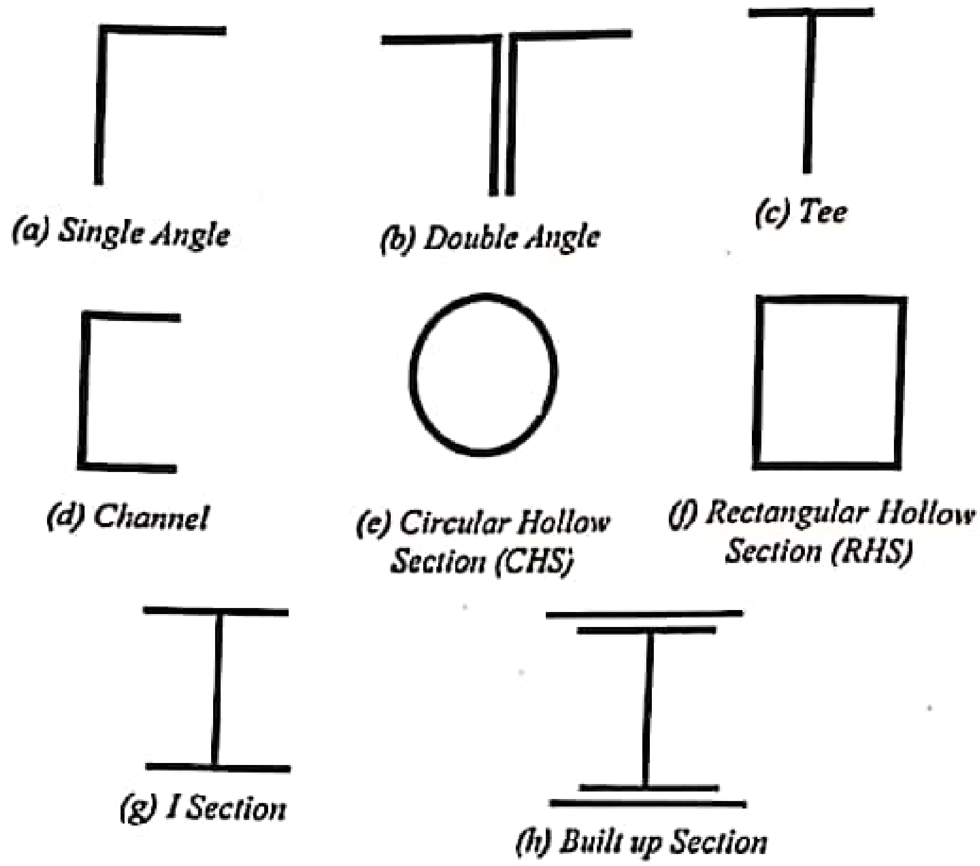


Fig. 6.1 Different Shapes for Rolled Steel Compression Members

6.3 : BUCKLING OF COLUMNS

Buckling is defined as the sudden bending, warping or crumpling of the compression members under compression.Due to buckling, deformation developed in a column occurs in a direction or plane normal to the direction of the loading. Buckling resistance depends on magnitude of the applied load, bending stiffness of the member and length of the member.Buckling of column is shown in Fig 6.2.

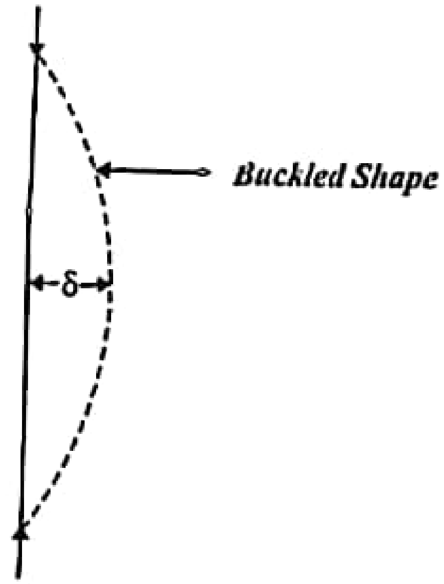


Fig. 6.2 Buckling of column

(i) **When end conditions can be assessed-** Where the boundary conditions in the plane of buckling can be assessed, the effective length, KL, can be calculated on the basis of Table 6.2.

(ii) **Compression member in trusses-** In case of bolted or welded trusses and braced frames, the effective length, KL, shall be taken as 0.7 to 1.0 times the actual length, depending upon the degree of end restraints provided. However, the effective length may be taken as actual length for buckling in the plane perpendicular to the plane of truss [Ref. Cl: 7.2.4 of IS 800 : 2007].

(iii) **Frames-** Where frame analysis does not consider the equilibrium of a framed structure in the deformed shape (Second-order analysis or advanced analysis), the effective length of compression members in such cases can be calculated using the procedure given in Appendix D.1. of IS: 800.

(iv) **Stepped columns-** The effective length of stepped column in individual buildings can be calculated using the procedure given in Appendix D.2. of IS: 800.

6.4.2 : *Appropriate radius of gyration*

The radius of gyration of member may be different about different transverse axes (YY, ZZ, UU, VV etc.). However, the radius of gyration of the compression member about the axis of buckling is known as appropriate radius of gyration.

6.4.3 : *Slenderness ratio*

It is defined as the ratio of effective length to the corresponding radius of gyration of the section. Thus $\lambda = \frac{l_e}{r} = \frac{KL}{r}$. The maximum slenderness ratio corresponding to minimum radius of gyration and / or maximum effective length, governs the design strength.

The maximum value of effective slenderness ratio for different member as per IS 800: 2007 is shown in Table 6.2. The effective length is cosidered as Table 6.3.

The mean compressive stress at buckling, f_{cr}, is given by

$$f_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 EI}{AL^2} = \frac{\pi^2 EAr^2}{AL^2} = \frac{\pi^2 Er^2}{L^2} = \frac{\pi^2 E}{\left(L^2/r^2\right)} = \frac{\pi^2 E}{\lambda^2}$$

where A = area of cross section of the compression member

r = radius of gyration of the cross section

L = effective length of compression member

λ = the slenderness ratio of the column defined by l/r

6.4 : EFFECTIVE LENGTHS OF COLUMNS AND APPROPRIATE RADIUS OF GYRATION

6.4.1 : *Effective length of column*

Effective length (l_e) of a column is defined in terms of equivalent length of the column hinged at both the ends for the various end conditions .

Both ends pin ended, $l_e = l$

Both ends fixed, $l_e = l/2$

One end fixed and the other end pinned or hinged, $l_e = \frac{l}{\sqrt{2}}$

One end fixed and the other end free, $l_e = 2l$

where l = actual length of the column.

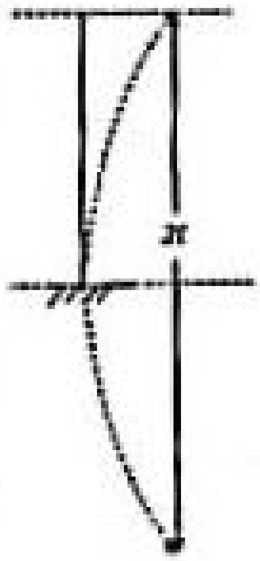


Fig. 6.3 Buckled mode for one end fixed and other end free

- As per IS: 800-2007, the effective length, KL, is calculated from the actual length, L, of the member, (with shilight modification from theoretical values taking into account practical considerations considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from center to center of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end, shall be taken as the actual length.

Design of Compressive stress and strength

The design compressive stress f_{cd} of axially loaded compression members shall be calculated,

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{m0}}$$

where, $\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$

λ = non-dimensional effective slenderness ratio.

$$= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

f_{cc} = Euler buckling stress $= \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

α = imperfection factor

$\gamma_{m0} = 1.1$ for Fe 415 steel.

The design compressive strength P_d of a member,

$$P_d = A_e f_{cd}.$$

where, A_e = effective sectional area, which is the same as gross area if bolt holes are filled with bolts.

imperfection factor → α.

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76.

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

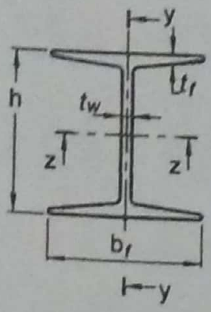
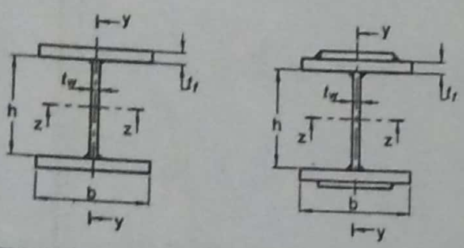
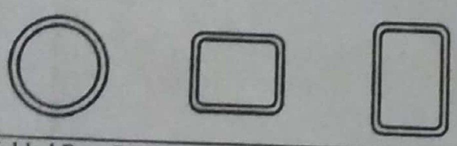
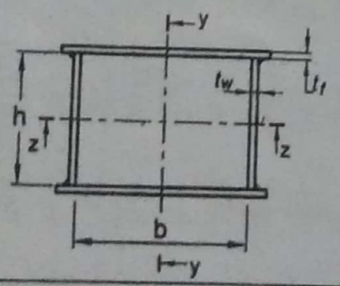
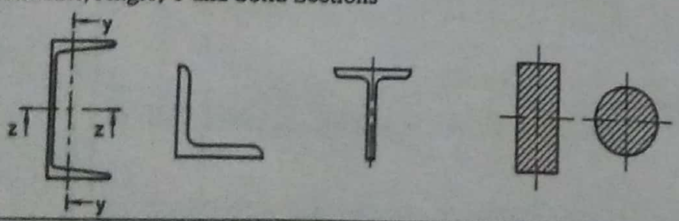
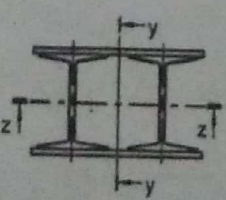
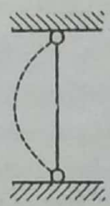
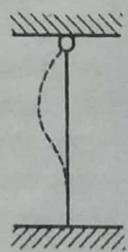
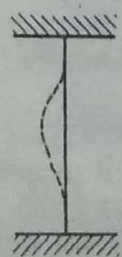
Cross-Section	Limits	Buckling About Axis	Buckling Class
(1)	(2)	(3)	(4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z y-y	c c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

Table 11 Effective Length of Prismatic Compression Members

(*Clause* 7.2.2)

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restrained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

NOTE — L is the unsupported length of the compression member (*see* **7.2.1**).

Q-1 Determine the design axial load capacity of the column ISHB 300 @ 577 N/m if the length of column is 3m and its both ends pinned.

Ans

For rolled steel sections,

$f_y = 250 N/mm^2$, $f_u = 410 N/mm^2$ and $E = 2\times10^5 N/mm^2$

For both end pinned columns

$KL = L = 3m$

For ISHB 300 @ 577 N/m

$h = 300mm$, $b_f = 250mm$, $t_f = 10.6mm$,

$A_e = A = 7484 mm^2$

$\therefore \frac{h}{b_f} = 1.2$ and $t_f < 40mm$

Hence according to Table 10 in IS 800, it falls under buckling class 'a' for buckling about z-z axis and under class b for buckling about y-y axis.

From steel table, $r_{min} = r_{yy} = 51.8mm$

Method-I

$$\therefore f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2\times 10^5}{\left(\frac{3000}{51.8}\right)^2} = 588.5 N/mm^2$$

Non-dimensionalised effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{588.5}} = 0.65177$$

For buckling class b,

$\alpha = 0.34$

$\therefore \phi = 0.5\left[1+\alpha(\lambda-0.2)+\lambda^2\right]$

$= 0.5\left[1+0.34(0.65177-0.2)+0.65177^2\right]$

$= 0.78920$

$\therefore f_{cd} = \dfrac{f_y/\gamma_{mo}}{\phi+(\phi^2-\lambda^2)^{0.5}}$

$= \dfrac{250/1.1}{0.78920+\left(0.78920^2-0.65177^2\right)^{0.5}}$

$= 184.14$

$\therefore$ strength of column, $P_d = A_e f_{cd}$

$= 7484 \times 184.14$

$= 1378138\,N = 1378.138\,KN.$ an.

Method - II (using Table No-9 - IS 800, page No - 36 → 43)

(i) About y-y axis

$\lambda_y = \dfrac{KL}{r_{yy}} = \dfrac{3000}{51.8} = 57.9151$ (effective slenderness ratio) < 180

For, $f_y = 250\,N/mm^2$ and buckling class b

By interpolation

KL/r	fcd
50	194
57.92	?
60	181

~~fcd = 194 − [(194−181)/(60−50)] ×~~

$$f_{cd} = 194 - \left[\frac{(194-181)}{(60-50)} \times (57.92-50)\right]$$

$= 188.456 \ N/mm^2$

$\therefore$ Factored axial load $= P_d = A_e \cdot f_{cd}$

$= 7484 \times 188.456$

$= 1410404.704 \ N.$

$= 1410.404 \ KN.$

(ii) About Z-Z axis

$$\lambda_z = \frac{KL}{r_{zz}} = \frac{3000}{129.5} = 23.17 < 180.$$

For $f_y = 250 \ N/mm^2$ and buckling class a.

By interpolation.

KL/r	fcd
20	226
23.17	?
30	220

$$f_{cd} = 226 - \left[\frac{(226-220)}{30-20} \times (23.17-20)\right]$$

$= 224.098 \ N/mm^2$

$\therefore$ Factored axial load $= P_d = A_e f_{cd}$

$= 7484 \times 224.098$

$= 1677149.432 \ N$

$= 1677.15 \ KN.$

$\therefore$ Design factored axial load = Min^n of the

$= 1410.404 \ KN.$ (Ans)

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: DESIGN OF AXIALLY LOADED COMPRESSION MEMBERS

The following procedure may be adopted in the design of axially loaded compression members.

1. Assume slenderness ratio and determine design compressive stress considering grade of steel and assuming buckling class. (The slenderness ratio may be considered as 70 to 90 for rolled steel beams, 110 to 130 for angle struts and 40 for members carrying large loads.) Alternatively the design stress in compression members may be directly assumed in the range of (a) 130 Mpa to 140 Mpa for rolled steel I - sections (b) 80 to 100 Mpa for angle struts and channels and (c) 190 to 200 Mpa for heavy / built up sections.
2. Calculate effective sectional area required $A_e = P_d / f_{cd}$
   Choose a trial section from steel table
   and Find r_{min} of the trial section
3. Find effective length and maximum slenderness ratio i.e. $\lambda_{max} = l / r_{min}$ considering end conditions and type of connections.
4. Determine permissible compressive stress f_{cd} considering grade of steel and actual buckling class and Compute the strength of the member $P_d = A_e f_{cd}$
5. Redesign if P_d differs considerably from the design load.
6. The section may be checked for limiting thickness also.
   Thus the most economical section can be arrived at by trial and error i.e. repeating the above process.

Example– 6.7: *Design a column section (using channel section only) to carry a factored axial load of 400 kN. The column is 4m long and is effectively held in position at both ends but restrained against rotation at one end only. Consider $f_y = 250$ N/mm². Assume wind / earthquake actions.*

Solution: *(All the clauses and tables mentioned in the solution refer to IS 800: 2007)*

(1) Assuming permissible design compressive stress 80 MPa

$$A_{reqd} = \frac{400 \times 10^3}{80} = 5000 \text{ mm}^2$$

(2) Try ISMC350 @ 413 N/m, having A = 5366 mm² (From steel table)

$r_{zz} = 136.6$ mm, $r_{yy} = 28.3$ mm

$r_{min} = r_{yy} = 28.3$ mm

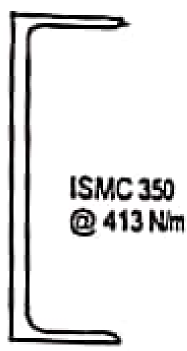
(3) For one end fixed and other end pinned,

$KL = 0.8\ L = 0.8 \times 4000 = 3200$ mm [Table – 11]

$$\lambda_{max} = \frac{KL}{r_{min}} = \frac{3200}{28.3} = 113.07 < 250$$ [Table – 3]

(4) The buckling class is 'c' for channel section [Table – 10]

For $\frac{Kl}{r} = 113.07$ and $f_y = 250\ MPa$



Fi.g 6.12

Permisible comp. stress $f_{cd} = 91.25\ N/mm^2$ (by interpolation) [Table – 9(c)]

(5) Design strength $P_d = Af_{cd} = 5366 \times 91.25 = 489.65\ kN > 400\ kN$ [Cl. 7.1.2]

Hence safe.

(6) Check for limiting thickness

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

From steel tables, $b_f = 100$ mm, $h = 350$ mm, $t_f = 13.5$mm, $t_w = 8.1$mm, $R_1 = 14$mm

Here $b = 100$mm, $d = h - 2(t_f + R_1) = 350 - 2(13.5 + 14) = 295$

For channel section, $\frac{b}{t_f} = \frac{100}{8.1} = 12.35 < 15.7\,\varepsilon = 15.7$,

$$\frac{d}{t_w} = \frac{295}{8.1} = 36.42 < 42\,\varepsilon = 42$$ [Table-2] OK

Q.

Calculate maximum axial load for a ISHB 350 @ 710.2 N/m and 4.0m high. The column is restrained in direction and position at both the ends. Use steel of grade Fe 415.

Q

Design a column section to carry a factored axial load of 500kN. The column is 3.6m long and is effectively held in position and restrained against rotation at both ends. Consider $f_y = 250 N/mm^2$

CHAPTER 7

Design of Steel Beams

7.1 : INTRODUCTION

/ A beam is a structural member subjected to bending. It is subjected to transverse loads normal to its axis. Beams of light sections that support floor construction are termed as joists. Horizontal beams spanning between the adjacent trusses are known as purlins. Lintel is a beam that spans over openings in buildings. Header is a beam framed to two beams at right angles and supports joists on one side of it. Beam that supports the headers is termed as trimmer. The beam supporting the stair steps is termed as stringer. Rolled I sections with and without cover plates are normally used for floor beams. Channel, tee and angle sections are used in roof trusses as purlins and common rafters. Laterally stable steel beams can fail only by (*a*) flexure (*b*) shear or (*c*) bearing, assuming that local buckling of slender components does not occur. These three conditions are the criteria for limit state of collapse for steel beams. Steel beams would also become unserviceable due to excessive deflection and this is classified as a limit state of serviceability.

7.2 : BASIC CONCEPTS OF PLASTIC THEOREY

To understand plastic theory, consider a rectangular shape steel beam, simply supported at both ends, subjected to a concentrated load at the centre. Consider the section at the midspan.

(a) When maximum bending moment (*M*) is less than yield moment (*My*), all the extreme fibres are stressed below yield point.

(b) With increase in load, maximum bending moment equals to the yield moment i.e. *M*=*My*. The stress in extreme fibres reach the value of yield stress and begins to yield.

(c) With further increase in load, the maximum bending moment lies between yield moment and plastic moment (*Mp*).

(d) Practically, all the fibres at the section reach the yield stress and the section become fully plastic. The moment corresponding to this state is called the plastic moment of the section.

Plastic moment may also be defined as the magnitude of the bending moment at which a plastic hinge is formed. Consider an arbitary section subjected to a plastic moment *Mp* as shown in Fig 7.1.

Considering the equilibrium condition $\sum H = 0$ (7.1)

Total force in compression =Total force in tension

$$\sigma_y A_t = \sigma_y A_c \text{ or } A_t = A_c \quad (7.2)$$

The neutral axis that divides the cross section into two equal halves is known as plastic neutral axis.

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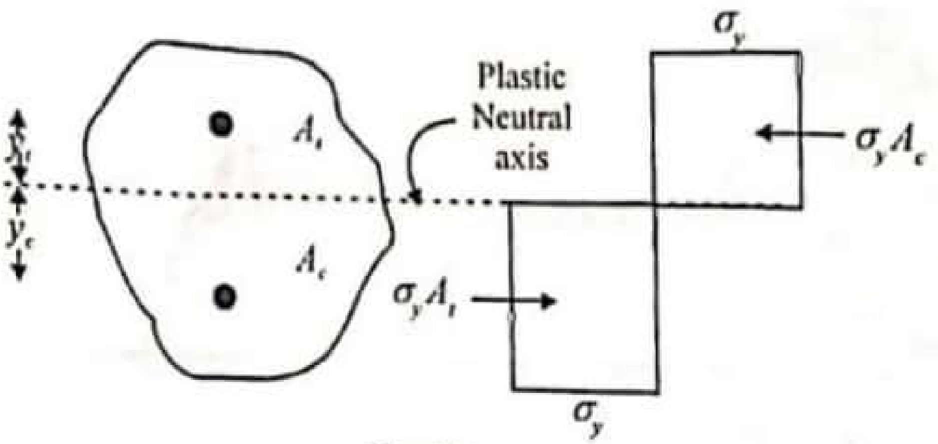

Fig. 7.1

$A_t = A_c = A/2$ (7.3)

Where A_t = Area in tension, A_c = Area in compression and A = Total area

Considering equilibrium condition $\sum M = 0$

$$M_p = (\sigma_y A_c) y_c + (\sigma_y A_t) y_t = \sigma_y (A_c y_c + A_t y_t) = \sigma_y Z_p \quad (7.4)$$

Where Z_p = Plastic section modulous = $A_c y_c + A_t y_t$ (7.5)

Shape factor of a crosss section is defined as the ratio between plastic moment to yield moment.

$$\text{Shape faxtor} = \frac{M_p}{M_y} = \frac{\sigma_y Z_p}{\sigma_y Z_e} = \frac{Z_p}{Z_e} \quad (7.6)$$

7.3 : LOCAL BUCKLING

Any plate element subjected to direct compression, bending or shear stress or combination of these stresses may buckle prematurely. Plate elements may fail in buckling locally before overall column buckling or overall beam failure due to yielding or lateral buckling. this reduces stiffness and strength of locally buckled plate elements. This type of failure is called local carryiong cpacity of columns and beams and distorts the cross section. This also decreases load doen not lead always to overall failure of the structure. Local buckling of any element width to thickness ratio of plate elements of different classes as given in Table 7.1. To prevent local buckling, IS code limits on [Table - 2, IS : 880]

7.4 : COMMON CROSS-SECTION AND THEIR CLASSIFICATION

The common cross section to be used as beam is shown in Fig. 7.2. Depending upon the slenderness of the constituent plate element of the beam, they are classified as slender, semi-compact, compact and plastic as shown in Table 7.1. Sections are also classified depending on their moment-rotation characteristics (Fig. 7.3). The code has specified the limiting width-thickness ratios for different sections.

- **Plastic cross-sections:** Plastic cross-sections are those which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism.

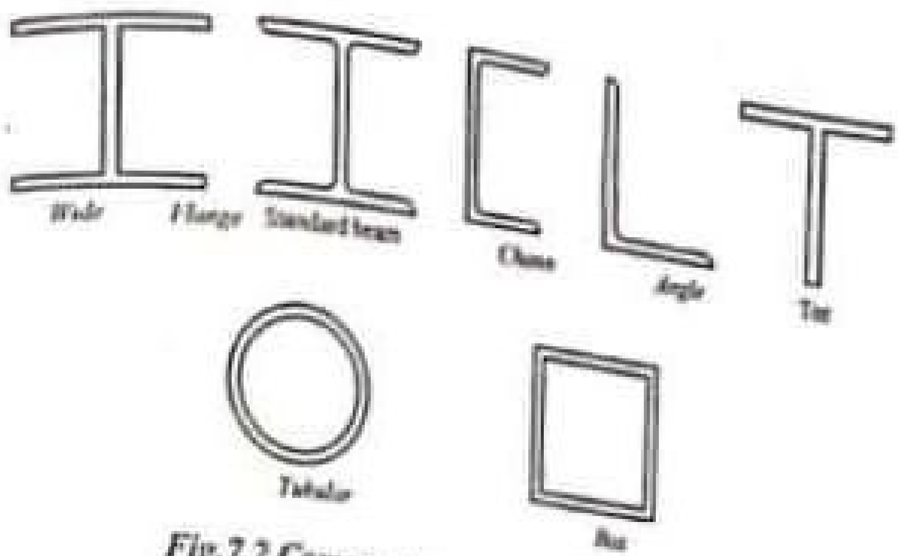

Fig.7.2 Common shape for beam section

- **Compact cross-sections:** Compact cross-sections are those which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling.
- **Semi-compact cross-sections:** Semi-compact cross-sections are those in which the stress in the extreme fibers in compression should be limited to yield stress. These sections can not develop the plastic moment of resistance due to local buckling.
- **Slender cross-sections:** Slender cross-sections are those in which the elements buckle locally even before reaching yield stress.

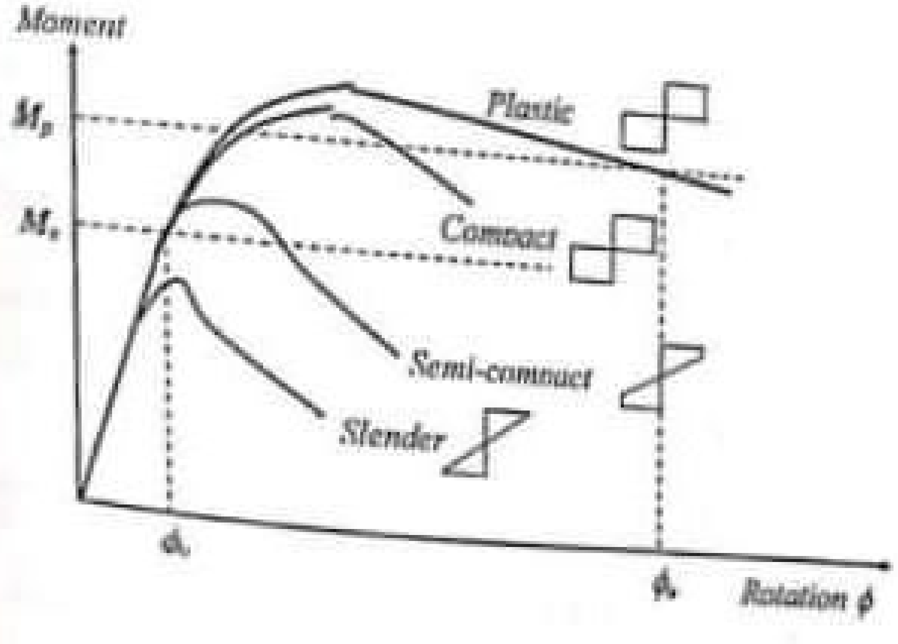

Fig. 7.3. Section Classification based on Moment-Rotation Characteristics

7.5 : DESIGN STRENGTH OF LATERALLY SUPPORTED BEAM IN FLEXURE (Codal Provisions IS 800:2007)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (*see Cl 8.2.1*). When a beam is not adequately supported against lateral buckling (laterally un-supported beams) the design bending strength may be governed by lateral torsional buckling strength (*see Cl 8.2.2*). The factored design moment, M at any section, in a beam due to external actions, shall satisfy $M \leq M_d$ (Cl 8.2.12)

where M_d = design bending strength of the section, calculated as given in *Cl 8.2.1.2*.of IS :800:2007

Laterally Supported Beam

A beam may be assumed to be adequately supported at the supports, provided the compression flange has full lateral restraint and nominal torsional restraint at supports supplied by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force. The design bending strength of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to *Cl 8.2.1.2*. (*Cl 8.2.1*)

When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section (see 8.4), the design bending strength, M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \qquad \textbf{[Cl 8.2.1.2]} \qquad (7.7)$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2 Z_e f_y / \gamma_{m0}$ in case of simply supported and $1.5 Z_e f_y / \gamma_{m0}$ in cantilever beams:

where

$\beta_b = 1.0$ for plastic and compact sections

$\beta_b = Z_e / Z_p$ for semi-compact sections;

Z_p, Z_e = plastic and elastic section modulii of the cross-section, respectively;

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (see 5.4.1).

When the design shear force (factored), V exceeds 0.6 V_d, where V_d is the design shear strength of the cross-section (see Cl 8.4) the design bending strength, M_d shall be taken as $M_d = M_{dv}$ Cl 8.2.1.3), where M_{dv} = design bending strength under high shear as defined in 9.2.

Combined Shear and Bending (*Cl : 9.2*)

No reduction in moment capacity of the section is necessary as long as the cross-section is not subjected to high shear force (factored value of applied shear force is less than or equal to 60 percent of the shear strength of the section as given in Cl: 8.4). The moment capacity may be taken as M_d (see Cl 8.2) without any reduction. **[Cl : 9.2.1]**.

When the factored value of the applied shear force is high (exceeds the limit specified in ...2.1), the factored moment of the section should be less than the moment capacity of the section ...high shear force, M_{dv} calculated as given below: [Cl 9.2.2]

(i) Plastic or Compact Section

$$M_{dv} = M_d - \beta\,(M_d - M_{fd}) \leq 1.2 Z_e f_y / \gamma_{m0} \tag{7.8}$$

$$\text{Where } \beta = \left(\frac{2V}{V_d} - 1\right)^2 \tag{7.9}$$

M_d = Plastic design moment of the whole section disregarding high shear force effect (see Cl: 8.2.1.2) considering web buckling effects (see Cl: 8.2.1.1),

V = Factored applied shear force as governed by web yielding or web buckling.

V_d = Design shear strength as governed by web yielding or web buckling (see Cl 8.4.1 or Cl 8.4.2),

M_{fd} = Plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{m0}, and

Z_e = Elastic section modulus of the whole section.

(ii) Semi-compact Section

$$M_{dv} = Z_e f_y / \gamma_{m0} \tag{7.10}$$

...: DESIGN STRENGTH OF LATERALLY SUPPORTED BEAM IN SHEAR (Codal Provisions)

The factored design shear force, V, in a beam due to external actions shall satisfy $V \leq V_d$ where V_d = design strength $= V_n / \gamma_{m0}$ [Ref : Cl : 8.4]

where γ_{m0} = partial safety factor against shear failure [see Cl 5.4.1].

The nominal shear strength of a cross section, V_n, may be governed by plastic shear resistance (Cl: 8.4.1) or strength of the web governed by shear buckling [Cl : 8.4.2].

The nominal plastic shear resistance under pure shear is given by:

$V_n = V_p$ [Cl 8.4.1]

$$\text{Where } V_p = \frac{A_v f_{yw}}{\sqrt{3}} \tag{7.11}$$

A_v = shear area, and f_{yw} = yield strength of the web

The shear area may be calculated as given below: [Cl 8.4.1.1]

I and channel sections:

Major Axis Bending :

Hot-Rolled — ht_w

Welded — dt_w

Minor Axis Bending:

Hot-Rolled or Welded — $2bt_f$

Rectangular hollow sections of uniform thickness:

Loaded parallel to depth *(h)* — $\dfrac{Ah}{b+h}$

Loaded parallel to width *(b)* — $\dfrac{Ab}{b+h}$

Circular hollow tubes of uniform thickness — $2A/\pi$

Plates and solid bars — A

where

A = cross-section area

b = overall breadth of tubular section, breadth of I-section flanges,

d = clear depth of the web between flanges,

h = overall depth of the section,

t_f = thickness of the flange, and

t_w =thickness of the web.

NOTE: Fastener holes need not be accounted for in plastic design shear strength calculation provided that :

$$A_{vn} \geq \left(\frac{f_y}{f_u}\right)\left(\frac{\gamma_{m1}}{\gamma_{m0}}\right) A_v / 0.9 \qquad (7.12)$$

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Section 9 of IS 800 maybe referred to for design strength under combined high shear and bending.

7.7 : WEB BUCKLING AND WEB CRIPPLING

7.7.1 : Web buckling : [Cl : 8.7.3]

Web buckling is the failure of web under the action of concentrated load. The web is subjected to column action during buckling. Buckling may occur below concentrated load or above a support. Web crippling is the failure of web in direct crushing under concentrated load. Web buckling and web crippling is shown in Fig 8.5 to 8.7. In both the cases, the load is spread out over a finite length of the web called as dispersion length as shown in Fig.8.6 and 8.7. The dispersion length is taken as $B = (b_1 + n_1)$ where b_1 is the stiff bearing length and n_1 is the dispersion of 45° line at the mid depth of the section. Web buckling strength is given by

$$P_{wb} = (b_1 + n_1)\, t_w\, f_{cd} \text{ where } n_1 = \frac{h}{2}, \text{ h = depth of the section.} \qquad (7.13)$$

where 't_w' is the web thickness and f_{cd} is the allowable compressive stress . The effective length of the strut is considered as $L_E = 0.7\,d$ where 'd' is the depth of the ***web*** in between the flanges determined as per codal provision. The slenderness ratio of the idealised web strut could be written as

$$\lambda = \frac{L_E}{r_y} = \frac{0.7d}{r_y} \qquad \text{Since } r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{t_w^3 B}{12 t_w B}} = \frac{t_w}{2\sqrt{3}} \qquad (7.14)$$

$\frac{l_e}{r_y} = 0.7d\frac{2\sqrt{3}}{t_w} \approx 2.5\frac{d}{t_w}$, d = depth of the web

Hence the slenderness ratio of idealised strut is approximately taken as $\lambda = 2.5\frac{d}{t}$. It may be further noted that at interior point where concentrated load is acting, dispersion n' will be twice that at support i.e. = $B' = (b_1 + 2n_1)$.

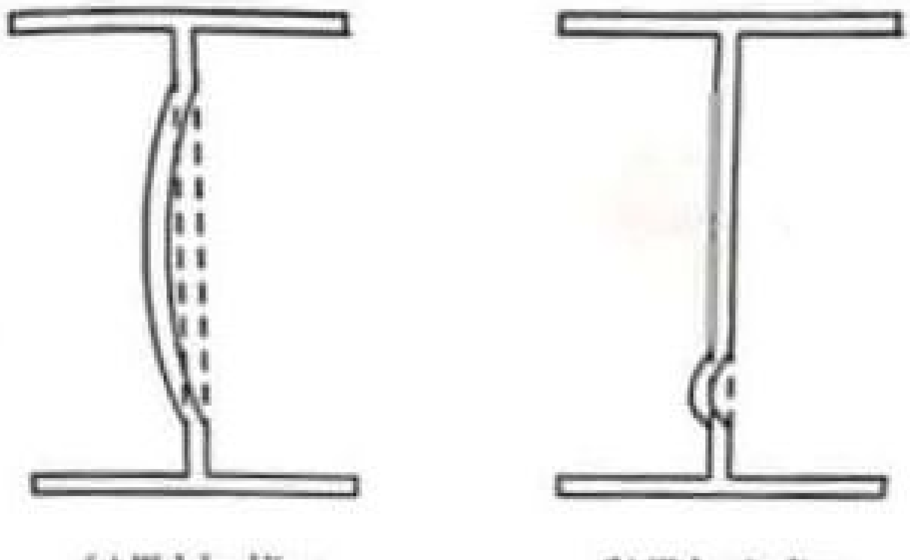

Fig. 7.5 Local buckling of Web

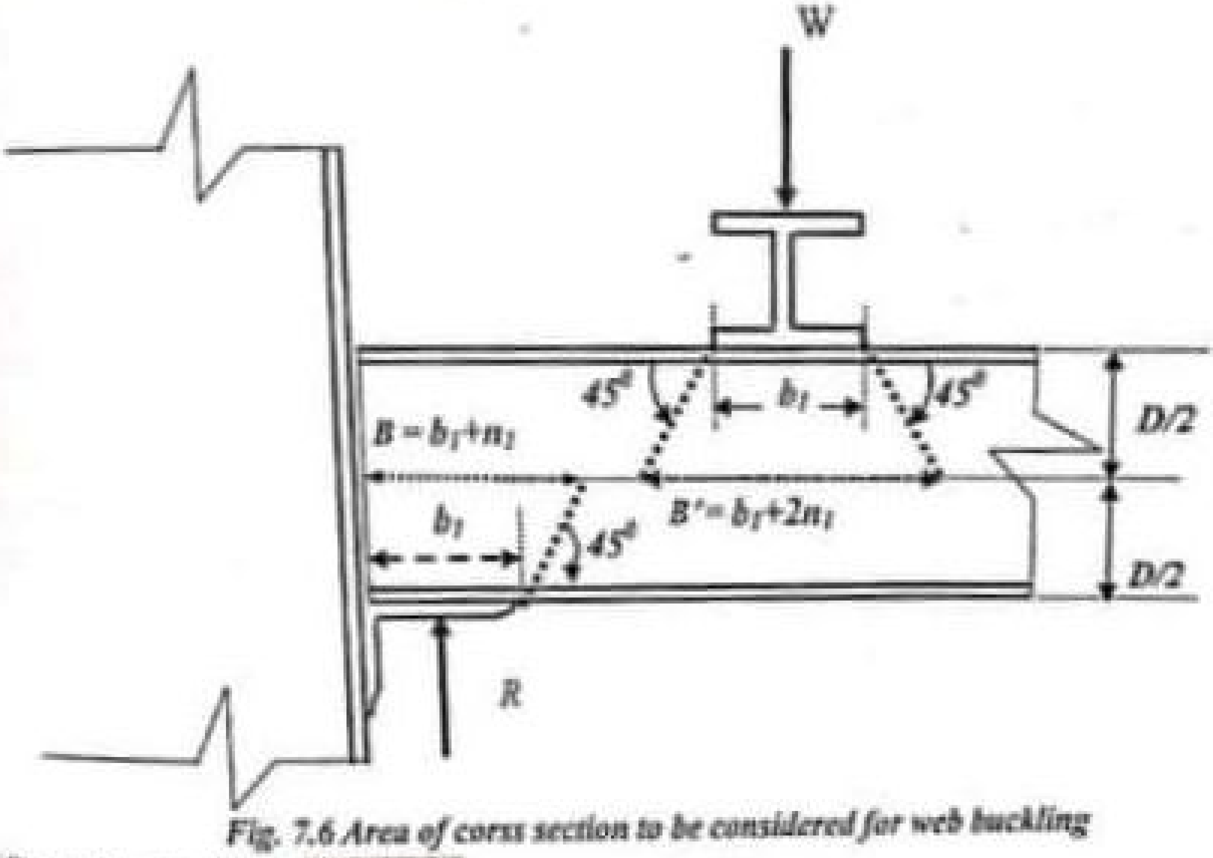

Fig. 7.6 Area of corss section to be considered for web buckling

7.7.2. Web crippling : [Cl : 8.7.4]

In case of web crippling, the crippling strength is calculated assuming a dispersion length of b_1+n_2, where n_2 is the length obtained by dispersion through the flange to web junction at a slope of 1:2.5 to the plane of the flange (i.e. $n_2 = 2.5d_1$) as shown in *Fig. 7.7.*

The crippling strength of the web at supports is calculated as (7.15)

$$P_{crip} = (b_1 + n_2)t_w\ f_{yw} / \gamma_{mo}$$

where f_{yw} is the design yield strength of the web.

where b_1 = Stiff bearing length

t_w = thickness of web

At an interior point, where concentrated load is acting, the crippling strength is given by, (7.16)

$$P_{crip} = (b_1 + 2n_2)t_w\ f_{yw} / \gamma_{mo}$$

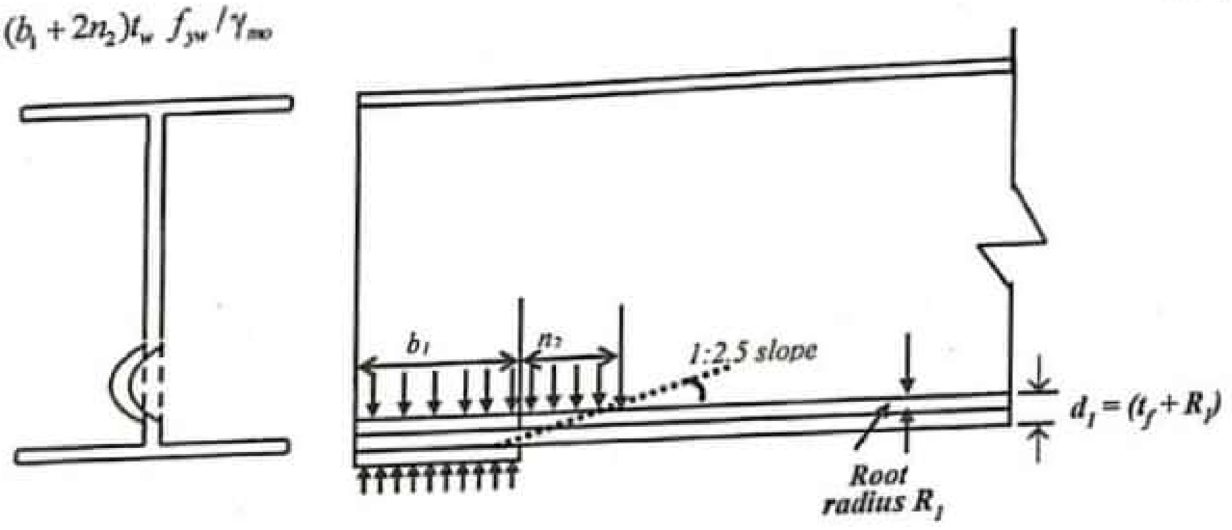

Fig. 7.7 Effective width of web bearing

7.8 : LIMIT STATE OF SERVICEABILITY – DEFLECTION

The deflection of a structure or structural component under serviceability loads shall not affect strength of the structure or components or cause damages to finishings. Excessive deflection in a floor of a building damages non stuctural components and causes discomfort and insecurity to the occupants in the building. Excessive deflection in industrial structures cause more vibration and damages to the machines. Deflections are to be checked by elastic analysis for the combination of service loads and their arrangements.The span to depth ratio (*L/d*) is only a guiding parameter to satisfy the Limit state of serviceability. The deflection limit for different structural members and systems are given in table 7.2.

7.9 : DESIGN PROCEDURE OF LATERALLY SUPPORTED BEAMS

The steps for design are as follows :

1. *Design moment and shear :*
   (a) Determine the service load on the beam and multiply with γ_f to find the ultimate load/ factored load.
   (b) Determine the effective span of the beam.
   (c) Calculate the maximum bending moment M and maximum shear force V.

2. *Trial section and section classification :*

(a) Determine a trial plastic section using the formula

$$Z_{p(reqd)} = M\gamma_{m0}/f_y$$

where M = Maximum bending moment determined in step -1 (c)

γ_{m0} = Yield stress of the material

f_y = Partial factor of safety

(b) From steel table, choose a trial section having plastic section modulous more than that required.

(c) Check the classification of section (see Table – 2 of IS 800)

3. *Check for bending and shear strength :*

(a) Determine shear strength of the section (Cl 8.4 of IS 800) and compare with maximum shear force V determined in step-1 (c).

(b) Find design bending strength depending on $V < 0.6V_d$ or $V > 0.6V_d$, as Cl : 8.2.1.2 or Cl: 9.2.2. of I.S. 800 :2007

Check M (maximum bending moment) < Design bending strength.

If not satisfied, repeat from step –2 (b) to 3 (b)

4. *Check for web buckling and web crippling :*

(a) Check the section for web buckling (Cl 8.7.3.1). The web buckling strength should be greater than end reaction at the support.

(b) Check for web crippling (Cl 8.7.4). The web bearing strength should be greater than load transferred by bearing i.e. reaction or concentrated load.

5. *Check for deflection :*

Check for deflection (Cl 5.6.1 & Table 6). Calculate the maximum deflection in the beam considering effective span, loading and support condition. The maximum deflection shall be less than the permissible value given in Table 6 of IS 800.

7.10 : *BUILT-UP BEAMS*

Built up beams are made up of two or more single rolled steel sections such as single I section with cover plates, two I sections with cover plates on both top and bottom flange, plate girders and gantry girders. Built up beams are used when a single rolled beam section is not capable of taking external moment or restriction of beam depth due to architectural cosiderations or the span is large. the cover plates shall be properly connected to rolled beam by bolting or welding. The outstands of cover plates shall be checked for slenderness ratio to prevent possibility of local buckling.

Example -7.1: ***Determine the plastic moment capacity and plastic section modulus of the rectangular section of size b × t about z-z axis as shown in fig 7.8***

Solution: Due to symmetry, plastic neutral axis (axis of equal areas) of the rectangular section is at mid depth

$$\therefore\ A_t = A_c = \frac{b}{2}\times t \qquad F = \frac{b}{2}t\,f_y$$

The distance between tensile and compressive forces $= \frac{t}{2}$

$$\therefore M_p = F \times \frac{t}{2} = \frac{b}{2} t f_y \frac{t}{2} = \frac{1}{4} bt^2 f_y$$

$$\therefore Z_p = \frac{M_p}{f_y} = \frac{1}{4} bt^2$$

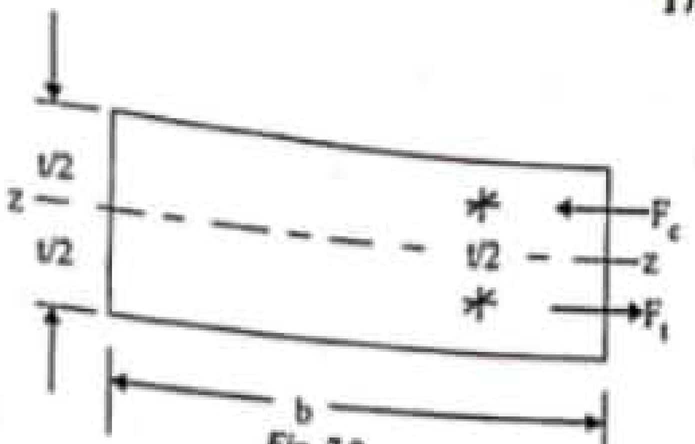

Fig. 7.8

...mple -7.2: ***Determine the plastic moment capacity and plastic section modulus of a symmetrical I section having depth of the section 350 mm, width of flange 140 mm, thickness of flange 14.2 mm and thickness of web 8.1 mm. (a) about z-z axis, (b) about y-y axis.***

...lution: (a) About z-z axis

Plastic N-A is at mid depth when plastic hinge is formed.

Forces in flanges, $F_1 = 140 \times 14.2 \times f_y$

Force in the webs, $F_2 = (175 - 14.2) \times 8.1 f_y$

Distance between F_1 forces $= 350 - 14.2 = 335.8$ mm

Distance between F_2 forces $= 2 \times \left(\frac{175 - 14.2}{2}\right) = 160.8$ mm

$$\therefore M_p = F_1 \times 335.8 + F_2 \times 160.8 = 140 \times 14.2 f_y \times 335.8 + 160.8 \times 8.1 f_y \times 160.8$$

$$= 667570.4\ f_y + 104719.392 f_y = 877009\ f_y$$

$$\therefore Z_p = \frac{M_p}{f_y} = 877.009 \times 10^3 \text{mm}^3$$

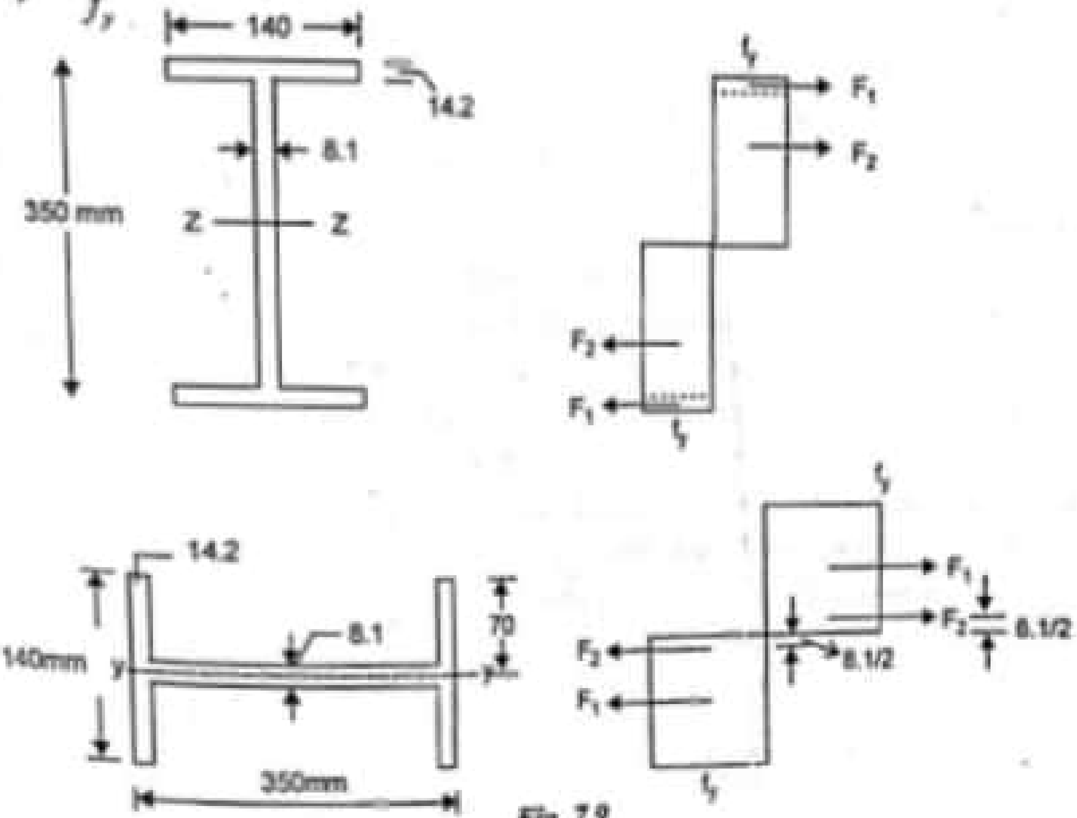

Fig. 7.9

(b) About y-y axis:

Plastic N-A is at the mid depth.

Let F_1 be force in flange of size 2 × 70 × 14.2 mm and F_2 be force in web of size 0.5 × (350 − 2 × 14.2) × 8.1 mm.

Hence, $F_1 = 140 \times 14.2 f_y$

$F_2 = 0.5 \times 321.6 \times 8.1\ f_y$

Distance between F_1 force $= \frac{140}{2} = 70$ mm

Distance between F_2 force $= \frac{8.1}{2} = 4.05$ mm

$\therefore M_p = 140 \times 14.2 \times f_y \times 70 + 0.5 \times 321.6 \times 8.1 f_y \times 4.05$

$= 139160 f_y + 5275.044 f_y = 144435.04\ f_y$

$\therefore\ Z_p = \frac{M_p}{f_y} = 144435.04\ \text{mm}^3$

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Example –7.8: ***Design a simple supported beam of clear span 1.2 m carrying a concentrated load 260 kN at mid span. Width of support is 200 mm. Consider f_y = 250 N/mm²***

Solution: *(All the clauses and tables mentioned in the solution refer to IS 800 : 2007)*

Load calculation:

Self weight (assumed) = 1 kN/m

Effective span = Centre to centre of supports = (1.2+ 2 × 0.2/2) = 1.4 m

Dead load = Self weight × Effective span

Factored load = working load × γ_f

$\gamma_f = 1.5$ [Table – 4]

∴ Total Factored load γ_f × (Dead load+super imposed load)

$= 1.5 \times (1 \times 1.4 + 260\ kN) = 392.1$ kN

...lation of design moment and shear:

Effective span of the simply supported beam = center to center distance of supports

Effective span = 1.2 + 0.2 = 1.4 m

As maximum bending moment occurs at a mid span for a simply supported beam with concentrated ...d at centre [Cl 8.1.1]

$\therefore$ Design moment, $M = 1.5\left[\frac{Wl}{4}+\frac{wl^2}{8}\right] = 1.5\left[\frac{260\times1.4}{4}+\frac{1\times1.4^2}{8}\right] = 136.87$ kNm.

Design shear force $V = 1.5\left[\frac{W}{2}+\frac{wl}{2}\right] = \frac{392.1}{2} = 196.05$ kN

$\gamma_{m0} = 1.1$ [Table – 5]

$\therefore$ Section modulus required $= \frac{M}{f_y}\times\gamma_{m0}$

$$Z_p = \frac{136.87\times10^6\times1.1}{250} = 602228\ \text{mm}^3$$

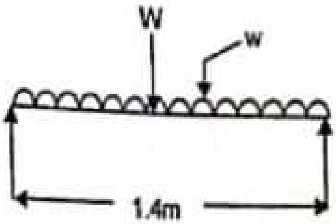


Fig. 7.17

Selection of trial section:

Try ISMB 350 @ 514 N/m having $Z_p = 889600\ \text{mm}^3$ (from steel table)

Depth of the section h = 350 mm

Width of flange b_f = 140 mm

Sectional area $A = 6670\ \text{mm}^2$

Thickness of flange t_f = 14.2 mm

Thickness of web t_w = 8.1 mm

Depth of web $d = h - 2(t_f + r_1)$

$= 350 - 2(14.2 + 14) = 293.6$ mm

Moment of inertia about z-z axis $I_{zz} = 13630.3\times10^4\ \text{mm}^4$

Elastic section Modulus $Z_e = 778.9\times10^3\ \text{mm}^3$ [Table – 2]

Section classification:

Here, $\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$

Out stand $b = \frac{b_f}{2} = \frac{140}{2} = 70mm$ [Fig. – 2]

Out stand of comp. flange for rolled section, $\frac{b}{t_f} = \frac{70}{14.2} = 4.93 < 9.4\varepsilon \Rightarrow$ Plastic section

Web of I-section with N. A. at mid depth, $\frac{d}{t_w} = \frac{193.6}{8.1} = 36.24 < 84\varepsilon \Rightarrow$ Plastic section

Hence the section is classified as plastic section. (least favourable classification)

Check for assumed self weight:

Weight of section = 0.514 kN/m < 1 kN/m (assumed) Hence O.K.

Check for shear strength: **[Cl : 8.4.1]**

Design shear $V = 196.05$ kN

Design shear strength of the section

$V_d = \frac{f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times$ Shear area [For rolled section, shear area = ht_w)

$= \frac{f_{yw}}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 350 \times 8.1 \text{ N} = 371997 \text{ N} = 371.997 \text{ kN} > 196.05 \text{ kN}$ Hence safe

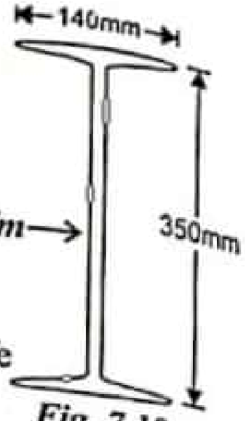


Fig. 7.18

Check for design moment capacity:

Here, $\frac{d}{t_w} = 36.24 < 67 \in = 67$ **[Cl : 8.2.1]**

$0.6V_d = 0.6 \times 371.997 = 223.198 \text{ kN} > 196.05 \text{ kN}$ **[Cl : 8.2.1.2]**

Hence, $M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}} < 1.2\, Z_e f_y / \gamma_{mo}$ (for s/s beam) **[Cl : 8.2.1.2]**

For plastic section, $\beta_b = 1$

$M_d = 1.0 \times 889.6 \times 10^3 \times 250 \times \frac{1}{1.1} = 202.18 \times 10^6$ N-mm

$= 202.18 \text{ kNm} < 1.2 \times 778.9 \times 10^3 \times \frac{250}{1.1} = 212.427 \times 10^6 \text{ Nmm} = 212.43 \text{ kNm}$

$\therefore M_d = 202.18 \text{ kNm} > 136.87 \text{ kNm}.$

Hence the section is safe

Check for deflection:

[Cl : 5.6.1 & Table – 6]

Permissible deflection for a beam in building assuming brittle cladding

$= \frac{l_e}{300} = \frac{1400}{300} = 4.67$ mm

Total working load = Dead load + live load = (0.514 × 1.4) + 260 = 260.72 kN = W'

Maximum deflection corresponding to working load

$$\delta = \frac{5wl^4}{384EI} + \frac{WL^3}{48EI} \approx \frac{W' \cdot L^3}{48EI}$$

$$\delta = \frac{W'L^3}{48EI} = \frac{260.72 \times 10^3 \times 1400^3}{48 \times 2 \times 10^5 \times 13630.3 \times 10^4} = 0.546 \text{ mm} < 4.67 \text{ mm}$$ Hence section is safe

Check for web buckling :

$\lambda_{effective} = 2.5\dfrac{d}{t_w} = 2.5\times36.24 = 90.6$

For buckling class is 'c' & $f_y = 250\ N/mm^2$

$f_c = 119 - \dfrac{0.6}{10}(119-105) = 118.16\ N/mm^2$

Dispersion width $n_1 = \dfrac{350}{2} = 175\ mm$

Web buckling resistance of the section, Assuming stiff bearing length $= b_1 = 50\ mm$

$\therefore$

$F_{wb} = (b_1 + n_1)t_w f_c$

$= (50+175)\times8.1\times118.16 = 215.346\times10^3 N = 215.346\ kN > 196.05\ kN$

Hence the section is safe.

Check for web crippling :

[Cl: 8.7.4]

Flange thickness = 114.2 mm, radius at root = 14 mm (From steel table)

$n_2 = 2.5\ (t_f + R_1) = 2.5(14.2+14) = 70.5\ mm$

$\therefore$ Strength of web against crippling

$F_w = (b_1 + n_2)t_w f_{yw} \times \dfrac{1}{\gamma_{m0}}$

$= (50+70.5)\ 14.2\times250\times\dfrac{1}{1.1} = 388.886\times10^3 N$

$= 388.886\ kN > 196.05\ kN$ i.e. load transferred by bearing $\Rightarrow$ safe

$\therefore$ Provide ISMB 350 @ 514 N/m as the beam section.

CHAPTER 8

Column Bases and Foundations

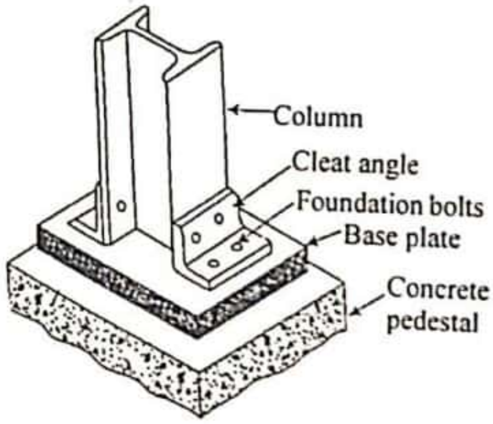
8.1. : *INTRODUCTION*

The load on column is transferred to the soil through column footing. The design compressive stress in a concrete footing is much smaller than that in a steel column. So it is necessary to provide a suitable base plate below the column to distribute the load from it evenly to the footing below. A column base spreads the load over a larger area so that the intensity of bearing pressure on the concrete block or masonry block does not exceed the permissible bearing stress. The column base shall be of sufficient strength, stiffness and area to transmit the load.

The column bases are of two types

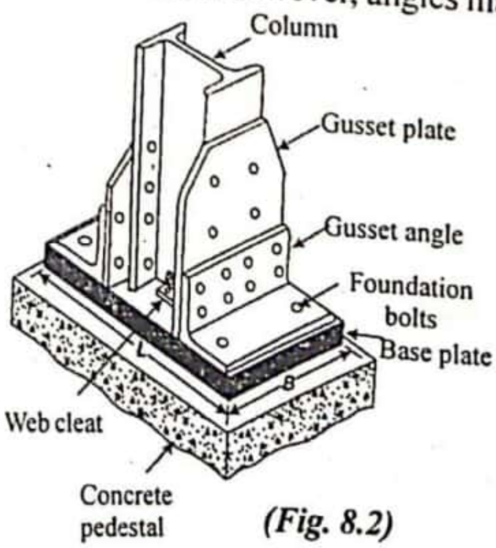
(a) Slab base (b) Gusseted base

(a) Slab base : When the column is subjected to small loads and the loading is mostly axial, a steel plate alone is used to transmitt loads to the concerete pedestal. Such a base plate is called slab base and is considered to be a pinned support. For purely axial loading, the base may be designed by assuming a uniform bearing pressure from below and a base plate directly attached to the column can be used to transmit the loads to the concrete pedesta. But when th columns are subjected to up lift or over turning forces, angle sections are attached to the flanges of the column which are anchored to the concrete pedestal through foundation bolts. Even if the column is subjeced to direct force only nominal angle sections called cleat angle are provided to keep the column in place and to resist any tension due to errection or accidental loads. However, angles may be omitted if base plate is directly welded to the column.



(Fig. 8.1)

(b) Gusseted base : For columns carrying heavy loads or axial load accompanied by bending moments, gusseted bases are used. In addition to base plate, it consits of gusset plates, gusset angles and cleat angles for bolted connections (angles may be omitted in case of welded connections). The gusset plates and angles are attached to the flanges of I - section where as cleat angles are used to connect web with base plate for correct positioning. Generally welded gusset bases are used for moderate loads. For same magnitude of load, the thickness of base plate for gusseted base in less than that of slab base due to the following reasons.



(Fig. 8.2)

(i) the gusset material used in the base increase the bearing area.

(ii) the gusset materials support the base plate against bending.

This kind of base may be considered to be rigid.

8.2. : COLUMN BASES

General Criteria :

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

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure should not exceed the bearing strength equal to $0.6f_{ck}$ where f_{ck} is the smaller of characteristic cube strength of concrete or bedding material. [Ref. Cl : 7.4.1. of I.S. 800 : 2007]

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, c, of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load Fig 8.3 (Fig 9 of IS 800), such that beam pressure on the effective area does not exceed bearing capacity of concrete base. [Ref. Cl : 7.4.1.1 of I.S. 800 : 2007]

Gusseted Bases :

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact. [Ref. Cl : 7.4.2. of I.S. 800 : 2007]

Where the ends of the column shaft and the gusset plates are not faced for complete bearing, the welding, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected. [Ref. Cl : 7.4.2.1 of I.S. 800 : 2007]

Column and Base Plate Connections :

Where the end of the column is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected. [Ref. Cl : 7.4.2.2 of I.S. 800 : 2007]

Slab Bases :

(i) Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. [Ref. Cl : 7.4.3]

(ii) Thinkness of slab base : The minimum thickness, t_s of rectangular slab bases, supporting columns under axial compression shall be

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

where,

w = uniform pressure from below on the slab ba[illegible] he factored load axial compression a, b = larger and smaller projection of the slab base beyo[illegible] ngle circumscribing the column, respectively

t_f = flange thickness of compression member

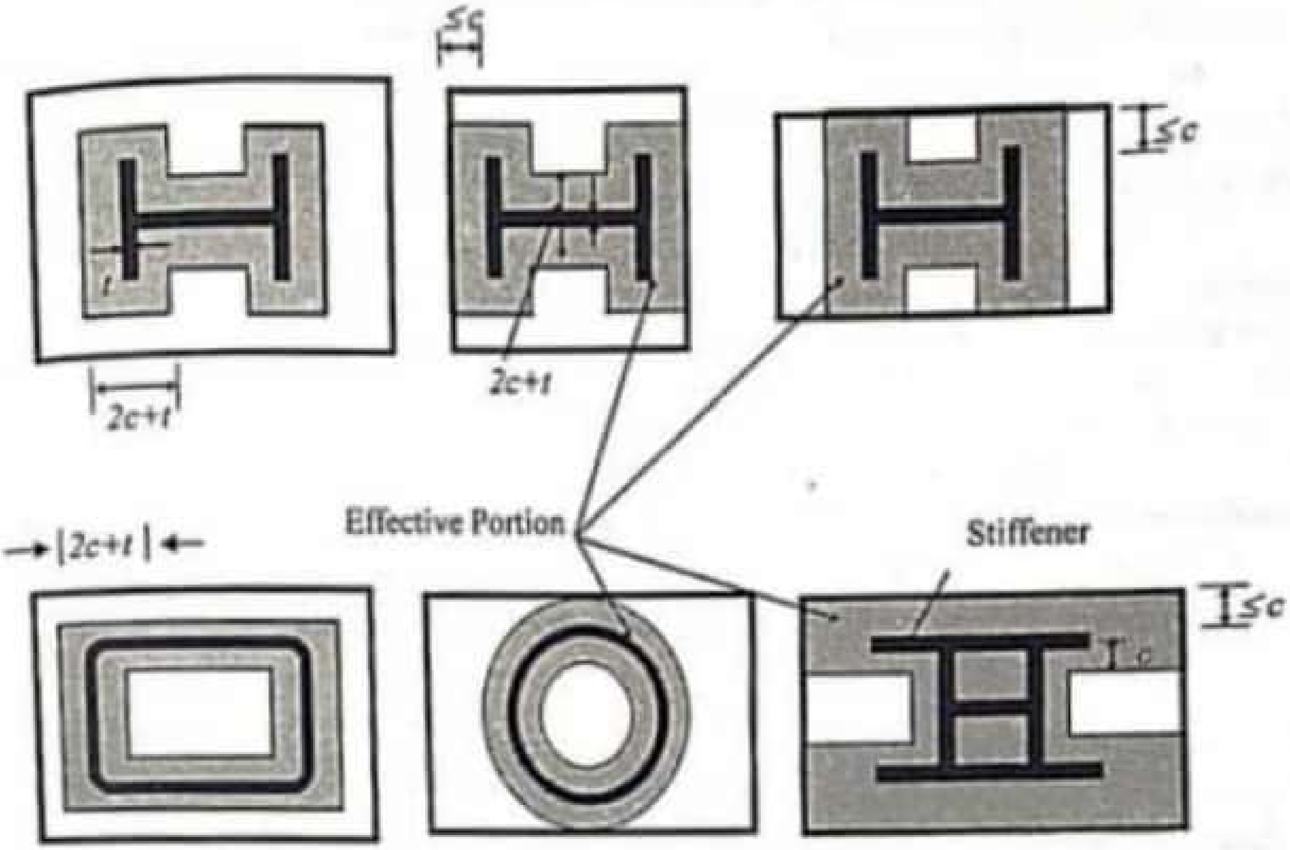


Fig. 8.3 (a) Effective Area of a Base Plate (Fig. 9 of IS 800)

When only the effective area of the base plate is used as in Cl : 7.4.1.1 of IS:800:2007 instead of (a^2-0.3b^2), c^2 may be used in the above equation (see Fig 8.3(a)) [Ref. Cl : 7.4.3.1 of IS : 800:2007]

When the slab does not distribute the column load uniformly, due to eccentricity of the load etc, special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below. [Ref. Cl : 7.4.3.2 of IS : 800 : 2007]

Bases for bearing upon concrete or masonry need not be machined on the underside. [Ref. Cl : 7.4.3.3 of IS : 800 : 2007]

In cases where the cap or base is fillet welded directly to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required in Cl : 7.4.3. Where full strength butt welds are provided, machining of contact surfaces is not required. [Ref. Cl : 7.4.3.4]

8.3 : GENERAL GUIDE LINES FOR COLUMN BASE

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, leveling 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 leveling 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 insuffici[...] [...]velop the applied bending moment or if thinner plates are used, some form of stiffening must [...]ed.

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.

8.4 : DESIGN OF SLAB BASE

Slab bases are suitable and economical for lightly loaded columns only. The thickness of base plate in decided from the consideration of bending of the portions of the base plate that extend bayond the column profile. Here the plate bending In two directions is taken into account. The moment in the direction of greater projection is reduced by the co-existent moment at right angles. Poissons ratio of 0.3 is used to allow for this effect. The design of slab bases with concentric load is covered in section 7.4.3 of IS 800 : 2007, where the rectangular plate is loaded by I, H, box or rectangular hollow sections. In cases where the projections are large or the loads are heavy, it may require double layer of plates or the use of stiffeners (vertical plates) to reduce the base plate thickness.

8.4.1 : *Design Procedure for Slab Base Subjected to Axial Loading*

The design of slab base consists in finding out its size and thickness. For axial loading pressure distribution is uniform under the slab base.

(A) Size of slab base :

1. Find maximum allowable bearing strength $= 0.45 f_{ck}$ depending upon grade of concrete.
2. Determine area of the base plate required by dividing factored load P_u with allowable bearing strength $0.45 f_{ck}$
3. Choose size of the base plate and find its area. For economy, as far as possible, the projections a and b are kept equal.

(B) Thickness of base plate :

4. Determine the actual intensity of pressure on concrete (w) by dividing factored load P_u with the actual area of the base plate. The actual intensity of pressure on concrete (w) shall be less than allowable bearing strength 0.45_{ck}
5. Determine base plate thickness $t_s = \sqrt{2.5 w (a^2 - 0.3 b^2) \gamma_{m0} / f_y}$

   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. 8.3 (b)]

   b = smaller plate projection from column

(C) Connections :

6. The column is suitably connected with the base plate.
   (a) If bolted connection is used for connecting the column to the base plate, two equal angles with suitable diameter of bolts are used.
   (b) If weld is used for connecting the column to base, fillet weld of suitable size and length is to be provided.
7. Connect the base plate to foundation concrete using 4 nos 20mm dia. and 300 mm long anchor bolts.

(i) the gusset material used in the base increase the bearing area.
(ii) the gusset materials support the base plate against bending.
This kind of base may be considered to be rigid.

8.2. : COLUMN BASES

General Criteria :

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

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure should not exceed the bearing strength equal to $0.6f_{ck}$ where f_{ck} is the smaller of characteristic cube strength of concrete or bedding material. [Ref. Cl : 7.4.1. of I.S. 800 : 2007]

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, c, of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load Fig 8.3 (Fig 9 of IS 800), such that beam pressure on the effective area does not exceed bearing capacity of concrete base. [Ref. Cl : 7.4.1.1 of I.S. 800 : 2007]

Gusseted Bases :

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc., in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact. [Ref. Cl : 7.4.2. of I.S. 800 : 2007]

Where the ends of the column shaft and the gusset plates are not faced for complete bearing, the welding, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected. [Ref. Cl : 7.4.2.1 of I.S. 800 : 2007]

Column and Base Plate Connections :

Where the end of the column is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected. [Ref. Cl : 7.4.2.2 of I.S. 800 : 2007]

Slab Bases :

(i) Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. [Ref. Cl : 7.4.3]

(ii) Thinkness of slab base : The minimum thickness, t_s of rectangular slab bases, supporting columns under axial compression shall be

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

where,

w = uniform pressure from below on the slab ba[illegible] the factored load axial compression a, b = larger and smaller projection of the slab base beyo[illegible]ngle circumscribing the column, respectively

t_f = flange thickness of compression member

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.

8.4 : DESIGN OF SLAB BASE

Slab bases are suitable and economical for lightly loaded columns only. The thickness of base plate in decided from the consideration of bending of the portions of the base plate that extend bayond the column profile. Here the plate bending in two directions is taken into account. The moment in the direction of greater projection is reduced by the co-existent moment at right angles. Poissons ratio of 0.3 is used to allow for this effect. The design of slab bases with concentric load is covered in section 7.4.3 of IS 800 : 2007, where the rectangular plate is loaded by I, H, box or rectangular hollow sections. In cases where the projections are large or the loads are heavy, it may require double layer of plates or the use of stiffeners (vertical plates) to reduce the base plate thickness.

8.4.1 : *Design Procedure for Slab Base Subjected to Axial Loading*

The design of slab base consists in finding out its size and thickness. For axial loading pressure distribution is uniform under the slab base.

(A) Size of slab base :

1. Find maximum allowable bearing strength $= 0.45 f_{ck}$ depending upon grade of concrete.
2. Determine area of the base plate required by dividing factored load P_u with allowable bearing strength $0.45 f_{ck}$
3. Choose size of the base plate and find its area. For economy, as far as possible, the projections *a* and *b* are kept equal.

(B) Thickness of base plate :

4. Determine the actual intensity of pressure on concrete (w) by dividing factored load P_u with the actual area of the base plate. The actual intensity of pressure on concrete (w) shall be less than allowable bearing strength 0.45_{ck}
5. Determine base plate thickness $t_s = \sqrt{2.5\,w(a^2 - 0.3b^2)\gamma_{m0} / f_y}$

   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. 8.3 (b)]

   b = smaller plate projection from column

(C) Connections :

6. The column is suitably connected with the base plate.
   (a) If bolted connection is used for connecting the column to the base plate, two equal angles with suitable diameter of bolts are used.
   (b) If weld is used for connecti[illegible] column to base, fillet weld of suitable size and length is to be provided.
7. Connect the base plate to found[illegible]oncrete using 4 nos 20mm dia. and 300 mm long anchor bolts.

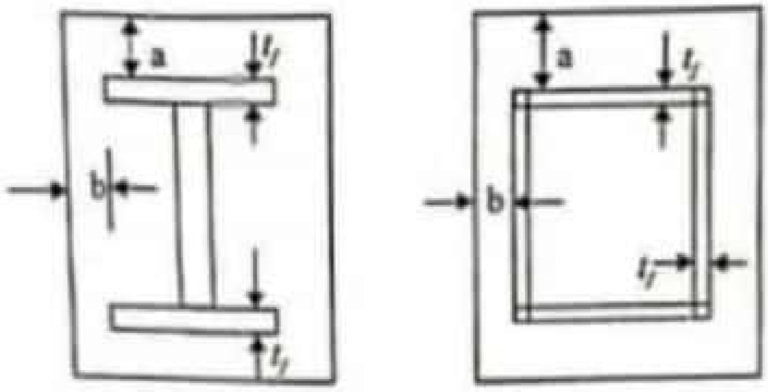


Fig. 8.3 (b) Base plates subjected to concentric forces

Example–8.1: ***A column section of ISHB400 @ 806.4N/m is to carry a factored load of 2000 kN. Use M25 grade of concrete for foundation, and Fe 410 grade of steel. Load is transferred to the base plate by direct bearing of column flanges. Design the slab base along with plain concrete footing. (SBC of soil = 200 kN/m².)***

Solution: [*All the clauses and tables refer to IS 800 : 2007 unless specified otherwise*]

(1) Area of base plate :

For M25 conrcete, f_{ck} = 25 Mpa.

Bearing strength of concrete = $0.45 f_{ck} = 0.45 \times 25 = 11.25$ N/mm² [Cl : 34.4 of IS 456:2000]

Area of base plate required = $2000 \times 10^3 / 11.25 = 177778$ mm²

Let us provided a base plate of 550 × 350 mm.

Area provided = 192500 mm² > Area required

(2) Thickness of base plate :

For ISHB 400, h = 400mm, b_f = 250 mm, t_f = 12.7mm (from steel tables)

$$\text{Projections are } a = \frac{L-h}{2} = \frac{550-400}{2} = 75 \text{ mm}$$

$$b = \frac{B-b_f}{2} = \frac{350-250}{2} = 50 \text{ mm}$$

$$\text{Pressure at base, } w = \left(\frac{2000 \times 10^3}{550 \times 350}\right) = 10.39 \text{ N/mm}^2$$

Now thickness of slab base, t_s

$$t_s = \sqrt{2.5 w (a^2 - 0.3 b^2) \gamma_{m0} / f_y} > t_f \qquad \text{[Cl : 7.4.3.1]}$$

$$= \sqrt{\frac{2.5 \times 10.39 (75^2 - 0.3 \times 50^2) \times 1.10}{250}} = 23.60 \text{ mm} > 12.7 \text{ mm} \Rightarrow \text{OK}$$

Since the available next higher thickness of plate is 25 mm, let us provide a base plate of size 550 × 350 × 25 mm.

(3) Connection of column to base plate :

As the load is transferred to the base pla ... ct bearing, it implies that the column end and base plate have been machined. Further, there i ... g moment. So the connection of column with base need not be designed. Two cleat angles of ... size 60 mm x60 mm × 8 mm may be provided to keep the column in position.

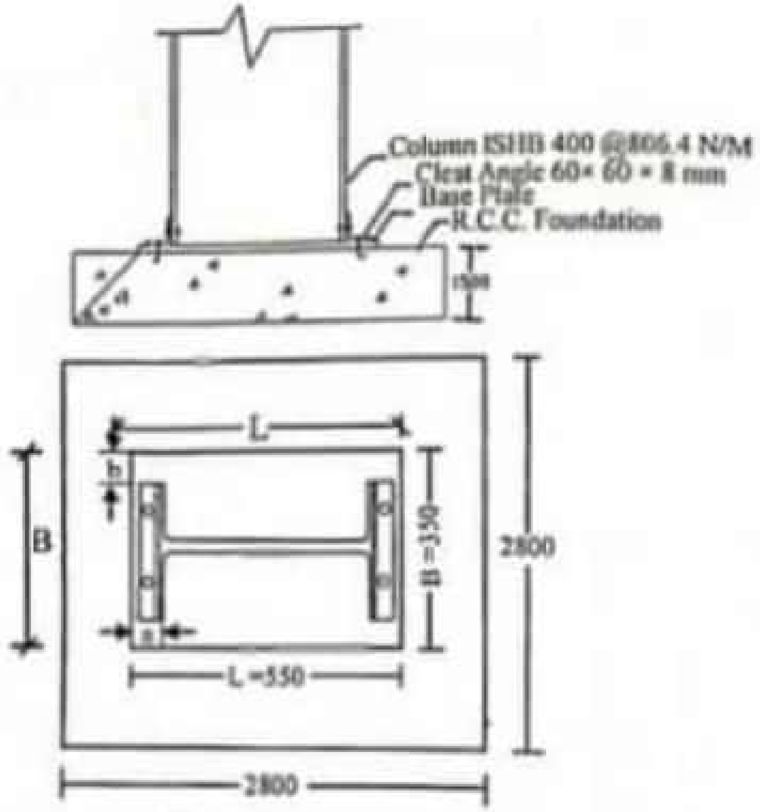


Fig. 8.4 Slab base.

(4) Connection of base plate to concrete founation :

Since the base is subjected to only axial compressive load and there is no bending moment, the base is not subjected to tension in any of its parts. Therefor, let us provide 4 nos. of nominal 20 mm diameter bolts, to keep the base in position.

(5) Concrete pedestal :

(a) Area of concrete of base

Factored Axial load = 2000 kN

Working load $= \frac{2000}{1.5} = 1333.33$ kN [Table – 4]

Assuming self weight of foundation @ 15%, total weight on soil

$= 1333.33 \times 0.15 = 200$ kN

Total load = (1333.33 + 200) KN = 1533.33 kN

Given safe bearing capacity of soil = 200 kN/m^2

Area of concrete base $= \frac{1533.33}{200} = 7.67$ m^2

Provide 2.8 m × 2.8 m

Area provided = 7.84 m^2 > 7.67 m^2

(6) Depth of concrete base :

Maximum bearing pressure at the [illegible] pedestal $q_0 = \frac{1533.33}{7.84} = 195.58$ kN/m^2

$= 195.58 \times 10^{-3}$ N/mm^2.

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If α is the angle of dispersion of load, $\tan\alpha \leq 0.9\sqrt{\frac{100q_0}{f_{ck}}+1}$ [Cl : 34.1.3. of IS 456 :2000]

or, $\tan\alpha \leq 0.9\sqrt{\frac{100\times195.58\times10^{-3}}{25}+1} = 1.20$ or $\alpha = 50.23°$

Maximum projection of concrete pedestal beyand slab base $p = \frac{2.8-0.35}{2} = 1.225\text{m}$

Depth of concrete pedestal, D = $p \tan\alpha$ = 1.225 × 1.2 = 1.47m. say 1.5m.

Hence let us provided a plain concrete pedestal of size = 2.8m × 2.8 m × 1.5 m

Example –8.2: ***A column ISWB 300@ 471.8 N/m is to carry an axial factored load of 800 kN. M20 concrete is used for the foundation. Design the slab base. Provide welded connection between column and base plate. Given that the column end and the base plate are not machined for bearing.***

Solution: [*All the clauses and tables refer to IS 800: 2007* unless otherwise specified]

(1) Area of base plate :

Bearing strength of concrete $= 0.45 f_{ck} = 0.45 \times 20 = 9$ N/mm^2 **[Cl : 34.4 of IS 456 : 2000]**

Factored load $P_u = 800$ kN

$$\therefore \quad \text{Area of base plate required} = \frac{800 \times 10^3}{9} = 88888.89 \text{ mm}^2$$

Let us provide 360 mm × 250 (L×B) size plate.

Area provided = 360 × 250 = 90000 mm$_2$ > 88888.89 mm^2 ⇒ OK

(2) Thickness of base plate :

$$\text{Base Pressure } w = \frac{800 \times 10^3}{90000} = 8.89 \text{ N/mm}^2$$

For ISWB 300 @ 471.8 N/m, h = 300 mm, b_f = 200mm, t_f = 10mm (from steel tables)

$$\text{Projections are } a = \frac{(L-h)}{2} = \frac{360-300}{2} = 30 \text{ mm}$$

$$b = \frac{(B-b)}{2} = \frac{250-200}{2} = 25 \text{ mm}$$

$$t_s = \sqrt{2.5\, w(a^2 - 0.3b^2)\gamma_{m0} / f_y} > t_f \qquad \textbf{[Cl : 7.4.3.1]}$$

$$\therefore \; t_s = \left[\frac{2.5 \times 8.89(30^2 - 0.3 \times 25^2) \times 1.1}{250}\right]^{0.5} = 8.124 \text{ mm} < t_f = 10\text{mm}$$

Hence provide t_s = 12 mm

∴ Size of base plate 360 mm × 250 mm × 12 mm.

(3) Connection of column to the base plate :

The column is to be connected to base plate using fillet weld.

For shop welding, γ_{mw} = 1.25 [Table – 5], f_u = 250 Mpa [Table –1]

Total length available for welding

= 2 [200 + 200 – 7.4 + 300 – (10×2)] = 1345.2 mm

Strength of weld $f_{wd}/mm = \frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37$ N/mm [Cl : 10.5.7.1.1]

Let s be the size of weld. Then the effective area of weld = 0.7 s L_e [Cl : 10.5.3.2]

when L_e is the effective length.

∴ The design condition is Effective area × strength of weld / mm = Pu

$0.7\, s\, L_e \times 189.37 = 800 \times 10^3$

$s\, L_e = 6035.05$

Minimum size of weld for 12mm plate = 5 mm.

Maximum of size weld = 10 – 1.5 = 8.5mm. [Cl : 10.5.8.1]

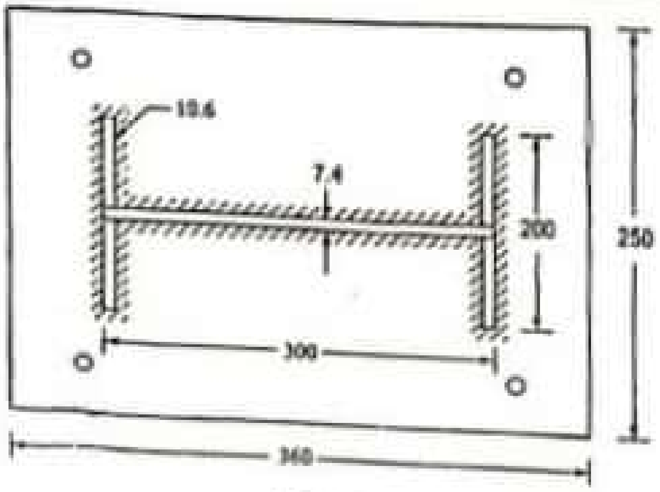

Fig. 7.5

Using 6 mm weld, L_e =1005.84 mm.

Available effective length after deducting for end return @ twice the weld size

= 1345.2 – 2 × 6 × No of Returns [Cl : 10.5.4.1]

=1345.2 – 2 × 6 × 12 =1201.2 > 1005.84 mm.

Hence 6 mm weld is adequate.

(5) Connection of base plate to the concrete foundation :

Use 4 anchor bolts of 20mm dia. and 300mm length nominally.

8.5 : DESIGN OF GUSSETED BASE

The base slab may be designed for bending at critical sections.

Moment of resistance (MR) = Maximum bending moment at the critical section (BM)

Maximum bending moment at $X - X = \frac{(w \times a^2)}{2}$

Maximum bending moment at $y-y = \frac{w(B-2b)^2}{8} - \frac{wb^2}{2}$

Where, w = actual intensity of pressure on concrete

B = Width of the base plate

a = cantilever projection beyond the face of gusset angle

b = Cantilever projection beyond the face of gusset plate.

The greater value of thickness obtained from two criteria is adopted.

The upward pressure from below the base causes bending of the gusset plates and puts their top edge in compression which is likely to buckle. This can be done by limiting the width to thickness ratio of guset plate so as to satisfy following criteria.

(i) For portion of the gusset plate welded between the column flanges, $D \le 29.3\ \varepsilon\ t_g$.

(ii) For out stand of the gusset plate from the egde of the column flange. So < $13.6\ \varepsilon\ t_g$ as a semi compact section.

The gusset plates may be designed to resist shear and bending. The moment in the gusset should not exceed the bending strength of the gusset plate given by $M_{dg} = \frac{f_{yg}}{\gamma_{mo}} \times Z_e$, where Z_e = elastic section modulus.

8.5.1 : Design Procedure for Gusseted Base Subjected to Axail Loading

The design procedure for gusseted base subjected to axial load with concrete footing is as follows.

(1) Approximate area of base plate

Find maximum allowble bearing strength of concrete = $0.45\ f_{ck}$ and determine area of the base plate required by dividing factored load P_u with allowable bearing strength $0.45\ f_{ck}$

(2) Selection of the gusset material

(a) Assume thickness of gusset plate (generally in the range of 14 to 20 mm).

(b) Assume size of gusset angle such that its vertical leg can accommodate two bolts in one vertical line and the other leg can accommodate one bolt. The thickness of gusset angle is approximately equal to the thickness of gusset plate. Generally, ISA 150 × 115 × 15 mm is used.

(3) Actual size of gusset plate

(a) Determine the width of the base plate (B) = width of column + 2x (thickness of cover plate + thickness of gusset plate + connected leg of gusset angle to base plate + projection of base plate out side the gusset angle).

(b) Determine length of the gus set plate (L) by dividing area of the base plate required to the width of the base plate (B). Find the area of the base plate provided i.e. L × B

(4) Thickness of base plate

(a) Determine the actual intensity of pressure on concrete (w) by dividing factored load P_u with the actual area of the base plate provided. The actual intensity of pressure on concrete (w) shall be less than allowable bearing strength $0.45\ f_{ck}$

(b) Compute the thickness of the base plate by flexural strength at the critical section i.e. one at the face of the vertical leg of the gusset angel X-X and the other being at the middle of the central portion Y-Y.

(5) Fastenings

(a) Design the connection between gusset plate and column for appropriate loading using either bolts or welds. The connection is designed for 50 percent of the design load transferred to both face of the column.

(b) Provide nominal fastenings for connecting cleat angle with column and base.

(6) Size of the gusset plate

The length of the gusset plate is kept equal to the side of the base plate farallel to which it is provided. The height of the gusset is governed by the number of rows of bolts or the length of the weld to be accomodated. The thickness of the gusset plate is checked for buckling and bending.

(7) Connect the base plate to foundation concrete by suitable number of anchor bolts.

Example –8.3: ***Design a gusseted base for a column ISHB250@537 N/m with two plates 400 mm × 20 mm carrying a working load of 2000 kN. The column is to be supported on concrete pedestal to be built with M20 concrete. Consider E250 (Fe 410 W).***

Solution: (*All the clauses and tables mentioned in the solution refer to IS 800 : 2007*]

(1) Area of base plate (approx)

Factored load =Working load $\times \gamma_f = 2000 \times 1.5 = 3000$ kN **[Table – 4]**

For concrete grade M20, $f_{ck} = 20$ N/mm² and bearing strength $= 0.45 f_{ck}$

[Cl : 34.4 of IS456:2000]

Area required $A = \dfrac{P_u}{0.45\, f_{ck}} = \dfrac{3000 \times 10^3}{0.45 \times 20} = 333333.3 \text{ mm}^2$

(2) Selection of gusset material

Selecting ISA 150115, 15 mm as gusset angle and 16 mm thick gusset plate

(3) Actual size of gusset plate

For ISHB 250 @ 537 N/m, h = 250mm $b_f = 250$ mm, $t_f = 9.7$mm, $f_y = 250$Mpa

Minimum width required $= 250 + 2 \times 20 + 2 \times 16 + 2 \times 115 = 552$ mm

Adopt B = 600 mm wide plate with projection on each side = (600 – 552) / 2 = 24 mm

$\therefore$ Required length of base plate L $= \dfrac{333333.300}{600} = 555.55$ mm

Let us provide 600mm × 600mm base plate.

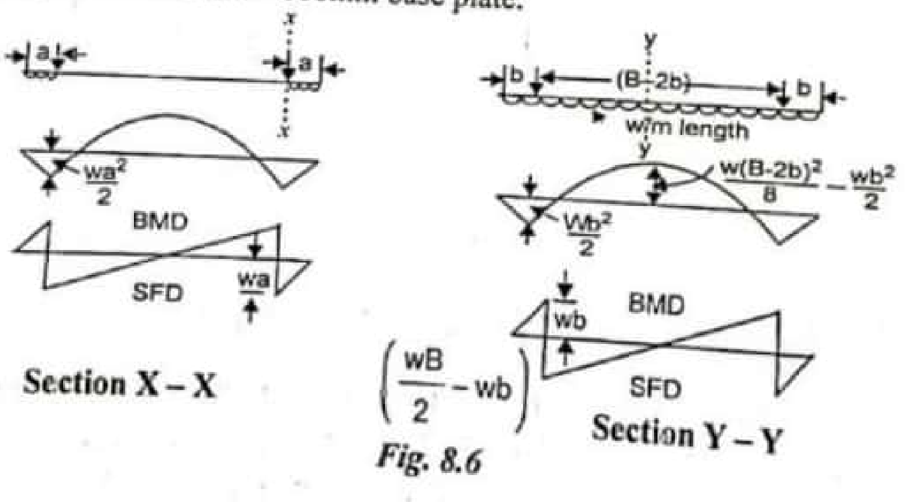

Fig. 8.6

(4) Thickness of base plate
Checked at two critical sections X - X & Y - Y

$\therefore$ Pressure under base plate $w = \frac{3000\times10^3}{600\times600} = 8.33\ N/mm^2$

(a) At section X - X (At the face of the gusset angle)

Cantilever projection, $a = \frac{600-(250+20\times2+16\times2+2\times15)}{2} = 124\ mm$

$\therefore$ BM at section X-X per mm width $= \frac{(w\times a^2)}{2}$

$M_{xx} = 8.33\times\frac{124^2}{2} = 64041.04$ N-mm

B.M. at section X-X will be resisted by by base plate and angle leg together. This portion may be considered as semi compact section, $\beta_b = Z_e/Z_p$.

M.R. $= M_d = \beta_b\ Z_p f_y/\gamma_{mo} = Z_e f_y/\gamma_{mo}$ {Here M_d = least of the two $= Z_e f_y/\gamma_{mo}$

For cantilever $< 1.5\ Z_e f_y/\gamma_{mo}$ [Cl : 8.2.1.2]

For unit width, M.R. = B.M.

$\Rightarrow \frac{1}{6}\times1(t+15)^2\times\frac{250}{1.1} = 64041.04 \Rightarrow t = 26.12mm$

Check for shear $V \not> 0.6Vd$ [Cl: 8.4]

Design shear V/mm $= w\frac{B}{2} - wb = w\left(\frac{B}{2}-b\right) = 8.33\left(\frac{600}{2}-139\right) = 1341.13N$

Shear strength V_d/mm $= \frac{A_v\cdot f_{yw}}{\sqrt{3}\gamma_{mo}} = \frac{1\times26.12\times250}{\sqrt{3}\times1.1} = 3427.36N$

$\therefore V = 1341.13 < 0.6\ V_d = 0.6\times3427.36 = 2056.4N \Rightarrow$ OK

(b) At section Y-Y, maximum net bending moment at mid span $M_{yy} = \frac{w(B-2b)^2}{8} - \frac{wb^2}{2}$

Where b = projection beyond face of gusset plate

$= \frac{600-(250+2\times20+2\times16)}{2} = 139mm$

$M_{yy} = \frac{8.33\times(600-2\times139)^2}{8} - \frac{8.33\times139^2}{2} = 27489$ N-mm

B.M. at section Y-Y well be resisted by base plate alone.This portion may be considered as compact section, $\beta_b = 1.0$ & M.R. $= M_d = \beta_b Z_p f_y/\gamma_{mo} = Z_p f_y/\gamma_{mo}$

For s/s case, $<1.2\ Z_e f_y/\gamma_{mo}$

For rectangular section $Z_p = 1.5 Z_e$

Hence $M_d = 1.2\ Z_e f_y / \gamma_{m0}$

Considering unit width and equating M.R. = B.M.

$$1.2\times\frac{1}{6}\times 1\times t^2\times\frac{250}{1.1}=27489 \text{ or } t = 24.59\text{mm.}$$

Check for shear.

V/mm = wa = 8.33 × 124 = 1032.92 N $< 0.6V_d \Rightarrow$ OK

Hence let us adopt thickness of base plate = maximum of above = 26.12 mm ≈ 28mm.

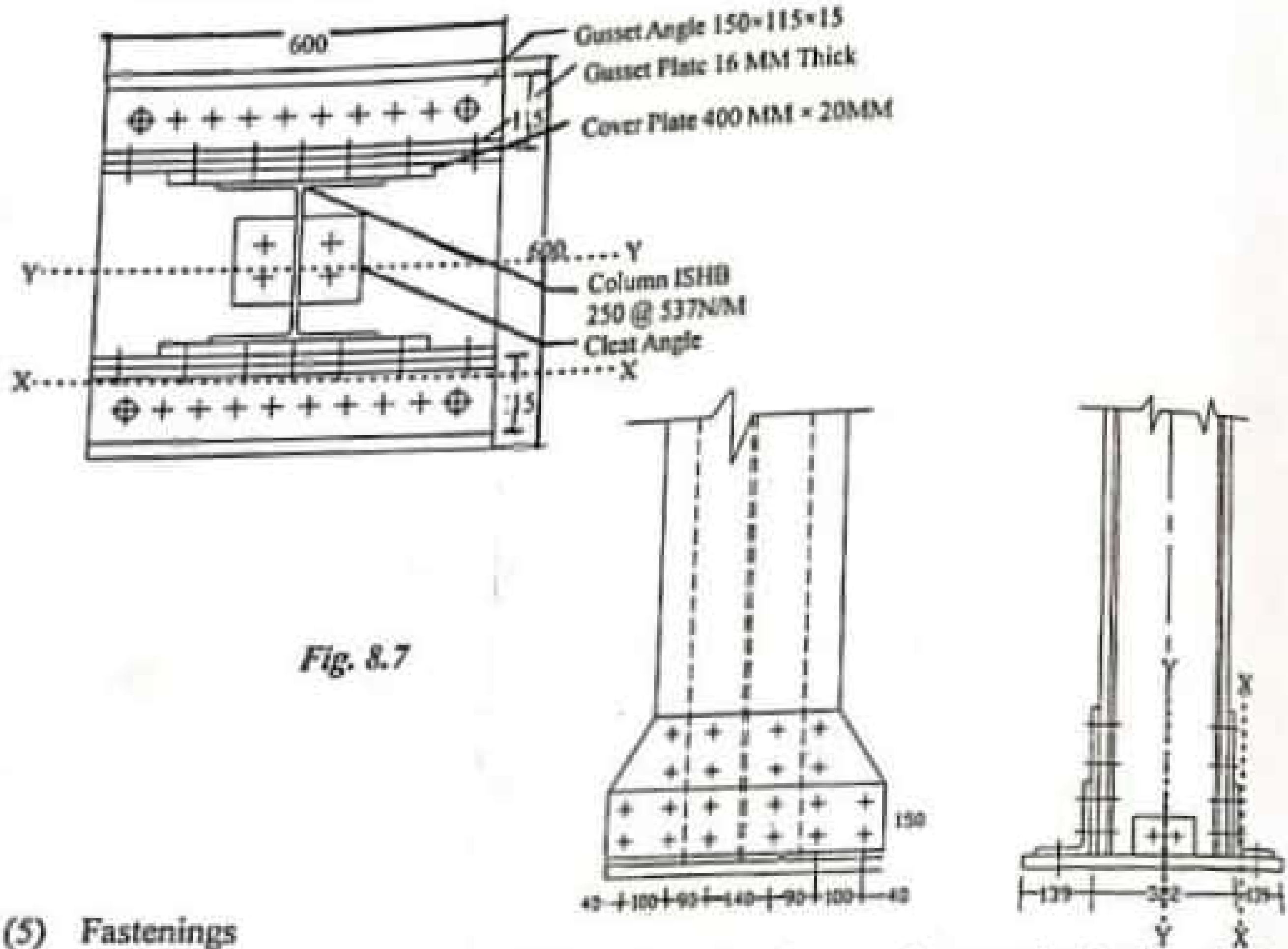

Fig. 8.7

(5) Fastenings

Part of the load is transfored to the base from the column and cover plate by direct bearing and the rest of the load is transfored to the gusset material at the face of the cover plates.

Projection beyond face of the cover plates = 600 – (250 + 2 × 20) = 310mm.

$$\text{Design load} = \frac{310}{600}\times 3000 = 1550 \text{ kN}$$

$$\text{Load on each face} = \frac{1550}{2} = 775 \text{ kN}.$$

Using 24 mm diameter shop bolts of **grade 4.6,**

$d_o = d_h = 24 + 2 = 26$mm, e = 39mm to **planned edge**

Strength of bolt in single shear $V_{dsb} = (n_n A_{nb} + n_s A_{sb})\frac{f_u}{\sqrt{3}}\beta_{lj}\beta_{lg}\beta_{pk}$ [Cl : 10.3.3]

Here, $n_n = 1.0$, $n_s = 0$, $f_u = 400\ MPa$, $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1.0$

$$= \left(1 \times 0.78 \times \frac{\pi}{4} \times 24^2 + 0\right) \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} \times 1 = 65192\ N$$

$$V_{dpb} = 2.5\, k_b dt \frac{f_u}{\gamma_{mb}} \text{ where } k_b = \text{least of } \begin{Bmatrix} \frac{e}{3d_0} = \frac{39}{3 \times 24} = 0.541 \\ \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 24} - 0.25 = 0.58 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \\ 1.0 \end{Bmatrix} = 0.541$$

$$= 2.5 \times 0.541 \times 24 \times 15 \times \frac{400}{1.25} = 155.81\text{kN}$$

Strength in bearing is higher.

∴ Bolt value = 65192 N.

$$\therefore \text{ No. of bolts required } = \frac{\text{Load on each face}}{\text{Bolt value}} = \frac{775 \times 10^3}{65192} = 11.89$$

Provide 12 numbers bolts of 24 mm diameter on each flange for connecting column to gusset plate along gauge line (g = 140mm). Connect the gusset plate and the angle with equal number of bolts as shown in the figure.

Minimum pitch = 2.5d = 2.5 × 24 = 60mm [Cl : 10.2.2]

Maximum pitch = 12t or 200mm for compression members [Cl : 10.2.3.2]

and = 100 + 4t or 200mm adjacent and parallel to edge of an outside plate. [Cl:10.2.3.3]

Here t = 15mm

Nominal number of 8 bolts may be used to connect cleat angle 2 ISA 9090,12mm to base plate.

(6) Gusset plate

Height of gusset plate = 150 + 2 × 39 + 1 × 60 = 288 mm ≈ 290mm

Length of gusset = width of base plate = 600mm

$$\text{Gusset out stand from the column} = \frac{600 - 400}{2} = 100\text{ mm}$$

$$\text{Averate height of the out stand for gusset angle} = \frac{0 + (290 - 150)}{2} = 70\text{mm}$$

$$\text{Here } \varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1 \text{ and } t_s = 16\text{mm}$$

$$\therefore 13.6\ \varepsilon t_s = 13.6 \times 1 \times 16 = 217.6\text{mm} \Rightarrow \text{OK}$$