(i) lap < $4t$

(ii) total length of weld at edge of batten < $\frac{D}{2}$

$$a + b + c < \frac{D}{2}$$

where, t = thickness of batten

Length of continuous weld at each edge of batten

< $\frac{1}{3}$ of total length required,

Return weld along transverse axis of the column < $4t$

where t and D are the thickness and the overall depth of battens, respectively,

Example: A column consisting of a steel channel section, ISMC 225 at 25.9 kgf/m, has an unsupported length of 3.5 m. It is effectively held in position and restrained against rotation at one end and restrained against rotation but not in position at the other end. Find the axial load the column can carry if the steel confirms to IS:226-1975.

Solution: Yield stress of steel as per IS : 226

$$= 250 \text{ N/mm}^2$$

Properties of ISMC 225 channel section:

Sectional area, $a = 33.01 \times 10^2$ mm^2

Radius of gyration, $r_{xx} = 90.3$ mm, $r_{yy} = 23.8$ mm

Effective length of column,

$$l = 1.2 \times 3.5 = 4.2 \text{ m} = 4200 \text{ mm}$$

Maximum slenderness ratio,

$$\lambda_{\text{maximum}} = \frac{l}{r_{min}} = \frac{4200.0}{23.8} = 176.50$$

From IS : 800, Permissible stress,

$$\sigma_{ac} = 33 + (37 - 33)\,\frac{3.5}{10} = 34.4 \text{ MPa}$$

Allowable axial compressive load

$$= \frac{34.4}{1000} \times 33.01 \times 10^2 = 113.6 \text{ kN}$$

Example : A single angle, ISA 60 × 60 × 6 mm of a roof truss is 1.50 m long. It isconnected by one rivet at each end. Find the safe load the strut can carry is

Solution : Sectional area, $a = 6.84$ cm^2

Least radius of gyration, $r = 1.15$ cm

Effective length = 1.50 m

Maximum slenderness ratio, $\lambda_{\max} = \frac{1.5 \times 100}{1.15} =$ 130.43

Assume yield stress of steel = 250 N/mm^2

$$\sigma_{ac} = 57 - (57 - 51) \times \frac{43}{10} = 56.74 \text{ MPa (N/mm}^2\text{)}$$

Allowable axial compressive stress for discontinuous single angle strut connected by single rivets at ends

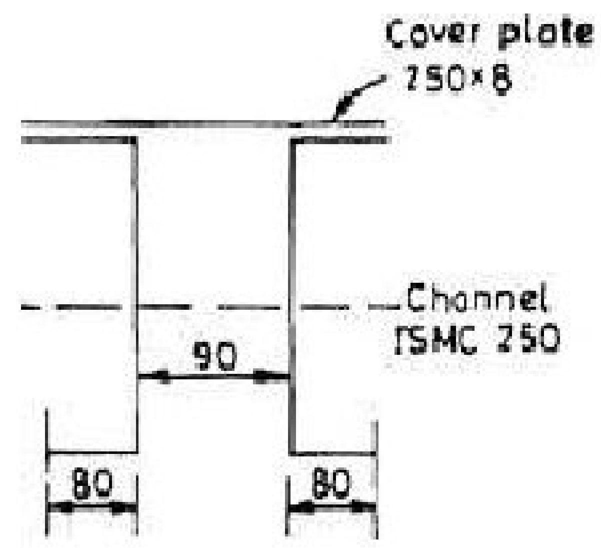
$$= 0.8 \times \sigma_{ac} = 0.8 \times 56.74 = 45.39 \text{ MPa}$$

Permissible axial load in compression

$$= \frac{45.394 \times 6.84 \times 100}{1000} = \mathbf{31.05 \text{ kN.}}$$

Example : The top chord of a bridge truss has an effective length of 5 m and cross-section as shown in the figure. The compressive load the member can carry if the yield stress of steel is 250 MPa, is

All dimensions in mm.



Solution: From steel tables for channel section, ISMC 250 at 30.6 kgf/m,

$$\text{Sectional area} = 38.6 \text{ cm}^2$$

Moment of inertia, $\quad I_{xx} = 3816.8$ cm^4

Distance of C.G. along *yy-axis* from web-face,

$$c_{yy} = 2.30 \text{ cm}$$

Cross-sectional area of chord member

$$= 2 \times 38.6 + 25 \times 0.8 = 97.2 \text{ cm}^2$$

Taking moments about top fibre, the depth *of e.g.* of built-up section from top

$$\bar{y} = \frac{(2 \times 38.6) \times (12.5 + 0.8) + 25 \times 0.8 \times 0.4}{97.2}$$

$$= 10.65 \text{ cm}$$

Moment of Inertia of the cross-section

$$I_x = 2 \times 3816.8 + 2 \times 38.67\,(13.3 - 10.65)^2$$

$$+ \frac{1}{12} \times 25 \times 0.8^3 + 25 \times 0.8\,(10.65 - 0.4)^2$$

$$= 10279.04 \text{ cm}^4$$

$$I_y = 2 \times 219.1 + 2 \times 38.67\,(4.50 + 2.30)^2 \times \frac{1}{12} \times 0.8 \times 25^3$$

$$= 5056.07 \text{ cm}^4$$

As $I_y < I_x$ and $r_y < r_x$

$$\therefore \qquad r_{\min} = r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{5056.07}{97.20}} = 7.21\text{cm}$$

Maximum slenderness ratio,

$$\lambda_{max} = \frac{l}{r_{min}} = \frac{500}{7.21} = 69.35, \text{ say, } 70$$

From IS : 800, σ_{ac} = 112 MPa

∴ Safe loading capacity of strut

$$= \frac{112}{1000}(97.20) \times 100 = \mathbf{1088.64\ kN}$$

DESIGN OF BEAMS AND BEAM COLUMNS

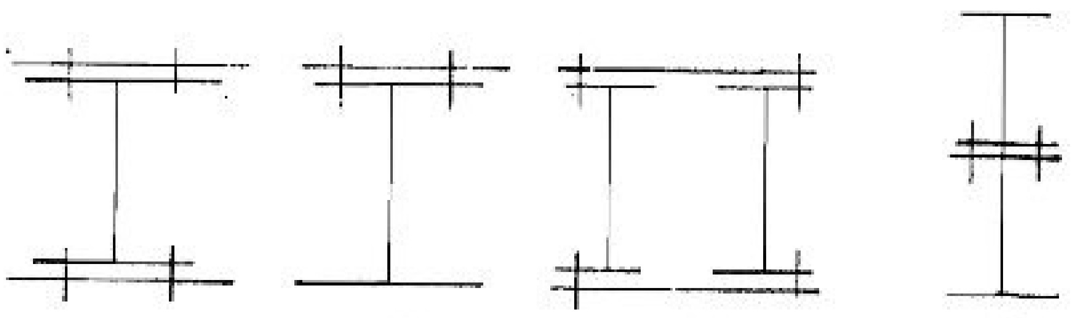
Beams transmit transverse loads normal to their axis to end supports through bending. Built-up beams made up of rolled sections and cover plates are used when

(i) the available rolled sections do not have adequate moment of inertia,

(ii) the beams have to be provided restricted-depth due to site conditions,

(iii) the spans are long, but the loads are not heavy.

The configurations generally used are shown in the figure below.



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

Design Basis

(a) Symmetrical built-up beams

The moment of inertia of a symmetrical built-up section shown in figure *(a)* to *(d)* is approximately given by

$$I = I_B + 2A_f \times \left(\frac{d}{2}\right)^2$$

where I_B = moment of inertia of rolled I-section,

A_f = area of each flange plate,

d = depth of the beam,

Dividing equation *(i)* by $\left(\frac{d}{2}\right)$, we get

$$Z = Z_B + A_f d$$

where Z_B = modulus of section of the beam, and

Z = modulus of section of the built-up beam,

$$A_f = \frac{(Z - Z_B)}{d}$$

The required built-up section can be chosen, using an I-section with a section modulus Z_B and flange plates having a net area of A_f each. The gross cross-sectional area of the flange plate is taken 20 per cent more than the net cross-sectional area, to allow for rivet holes and approximations in calculations.

(b) Unsymmetrical built-up beams

The area of cover plate may be calculated approximately by

$$A_f = \frac{(1.20)(Z - Z_B)}{d}$$

After selecting the size of the cover plate, the neutral axis should be determined forthe compound section for evaluating the fibre stresses.

Rivet Connections

The rivets connecting cover plates to flanges are designed to resist the longitudinal shear stress τ_v at flange plate level.

$$\tau_v = \frac{V}{Ib}(A\bar{y})$$

or the horizontal shear per unit (centimetre) length of beam is,

$$\tau_v \times b \times 1 = \frac{V}{I}(A\bar{y})$$

where V = shear force (vertical) at the section considered,

I = moment of inertia of the compound section,

$A\bar{y}$ = moment of area of plate above flange of I-section about the neutral axis.

$$= b \times t \times \left(\frac{d}{2} + \frac{t}{2}\right)$$

where b = width of the flange plate, and t is the thickness of the flange plate.

If p is the pitch of rivets, then horizontal shear force per pitch length

$$= \frac{V(A\bar{y})p}{I} = \frac{V.d.t\left(\frac{d}{2} + \frac{t}{2}\right).p}{I}$$

If the rivet value is R and n rivets are in a pitch length, then

$$n \times R = \frac{V.d.t\left(\frac{d}{2} + \frac{t}{2}\right)}{I}$$

orpitch of rivets, $$p = \frac{d \times R \times I}{V \times b \times t \times \frac{d+t}{2}}$$

The pitch of rivets provided should not exceed the value p given by above equation. The outstand of the flange plate beyond rivets in compression and tension should not exceed 16t and 20 t, respectively, for steel conforming to IS : 226.

Plate Thickness

The outstand of cover plates, flange plates, angle legs, etc. should be within the limits given in the Table to avoid local buckling.

	Type	Width of plate section	Maxium width
(i)	Flanges and plates in compression, unstiffened.	Flange-half the nominal width of flange of beam and tee sections.	$\frac{256t}{\sqrt{f_y}}$ subject to a maxium of 16t
(ii)	Flanges and plates in compression, stiffened.	Plates- distance from the tree edge to the first row of rivets or welds.	20t
(iii)	Flanges and plates in tension	From edge to the innermost face of the stiffener.	20t

t = thickness of a angle plate or total thickness of two or more plates effectively tacked together.

Where a plate is connected to other parts of a built-up member, the width between any two adjacent lines of connections or supports should not exceed the following:

(i) for plates in compression – $\frac{1440t}{\sqrt{f_y}}$ subject to a maximum of 90 t. However, the width should not exceed $\frac{560t}{\sqrt{f_y}}$ subject to a maximum of $35t$ for welded plates which are not stress relieved, or $\frac{800t}{\sqrt{f_y}}$ subject to a maximum of 50 t. For other plates, the excess width should be assumed to be located centrally and its sectional area neglected when calculating the effective geometric properties of the section.

(ii) for plates in uniform tension - 100 t, however, the excess width over 60t should be assumed to be located centrally and its sectional area neglected when calculating the geometrical properties of the section.

Check for Stresses

The built-up section should be checked for bending stresses in extreme fibres. The stresses are first worked out on the basis of the gross moment of inertia. Then the maximum flexural stresses are determined by increasing the stresses arrived at on the basis of gross moment of inertia in the ratio of gross area to the effective area of the flange section. The effective area of the compression flange should be the gross area with deductions for excessive width of plates and for open holes occurring in a plane perpendicular to the direction of stress at the section considered. The effective area of tension flanges should be the gross cross-sectional area of the flange with deduction for rivet holes.

The section should also be checked for maximum shear stress, considering the whole area of the cross-section, having regard to the actual distribution of shear stress.

Permissible Bending Stresses

Maximum permissible bending stress in tension is limited by

$$\sigma_{bt} = 0.66 f_y, \ (\sigma_{bt} \text{ and } f_y \text{ in MPa})$$

For beams bent about the axis of maximum strength (x-x axis), the permissible bending stress in compression is given by

$$\sigma_{bt} = 0.66 \left\{ \frac{f_{cb} \cdot f_y}{\left[(f_{cb})^n + (f_y)^n \right]^{1/n}} \right\}$$

where f_{cb} = elastic critical stress in bending (MPa),

f_y = yield stress of steel (MPa), and

n = a factor assumed as 1.4

The elastic critical stress, f_{cb}, can be determined by elastic flexural-torsional buckling analysis. In the absence of such an analysis, f_{cb} can be calculated for girders with I_y smaller than I_x, using the formula,

$$f_{cb} = k_1(X + k_2 Y)\frac{c_2}{c_1}$$

Where, $$X = Y\sqrt{1 + \frac{1}{20}\left(\frac{Tl}{r_y D}\right)^2} \text{ MPa,}$$

and $$Y = \frac{26.5 \times 10^5}{\left(\frac{l}{r_y}\right)^2} \text{ MPa,}$$

k_1 = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraint,

= effective length of compression flange,

r_y = radius of gyration of the section about its axis of minimum strength (y-y axis),

T = mean thickness of compression flange, equal to the area of the horizontal portion of flange divided by width,

D = overall depth of beam,

k_2 = a coefficient to allow for the inequality of flanges,

c_1, c_2 = the lesser andgreater distances from the section neutral axis to the extreme fibres respectively,

I_y = M.I. of the whole section about the axis lying in the plane of bending (y-y axis),

1_x = M.I. of the whole section about the axis normal to the plane of bending (x-x axis).

The value of k_1 is dependent on another factor, Ψ, defined as the ratio of both flanges at the point of least bending moment to the corresponding area at the point of the greatest bending moment between points of effective lateral restraint, value of k_l is equal to 1 if there is no curtailment of flanges.

The value of k_2 is dependent on a factor, ω, defined as the ratio of the moment of inertia of the compression flange alone to that of the sum of the moment of inertia of the flanges, each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment.

Values of f_{cb} should be increased by 20 per cent, when $\left(\frac{T}{t}\right)$ is not greater than 2.0 and $\left(\frac{d_l}{t}\right)$ is not greater than $\frac{1344}{\sqrt{f_y}}$.

where d_l = appropriate depth of web depending on provision of horizontal stiffeners,

and t = thickness of web.

Maximum permissible bending stress in tension σ_{bt} or in compression σ_{bc} for beams bent about the axis of minimum strength (y-y axis) should not exceed $0.66 f_y$, where f_y is yield stress of steel.

Permissible Bearing Stress

The bearing stress in any part of a beam when calculated on the net area of contact should not exceed the value of σ_p given by

$$\sigma_p = 0.75 f_y$$

Where σ_p = maximum permissible bearing stress, and

f_y = yield stress of steel.

Permissible Shear Stress

The maximum shear stress in a member, having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, should not exceed the value given by

$$\tau_{vm} = 0.45 f_y$$

where τ_{vm} = maximum permissible shear stress, and

f_y = yield stress of steel.

Effective Span

The effective span of a beam should be taken as the length between centres of supports, except in cases where the point of application of the reaction is taken as eccentric to the support, when it is permissible to take the effective span as the length between the assumed points of application of reaction.

Effective Length of Compression Flange

When lateral deflection of the compression flange is prevented by providing lateral support, the beam is said to be laterally supported. The effective lateral restraint is the restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a beam from lateral buckling to either side at the point of application of the restraint. In fully restrained beams, permissible compressive stress in bending is taken as equal to the permissible bending tensile stress.

In laterally unsupported beams, the permissible compressive stress in bending, f_{bc} is dependent on the elastic critical stress in bending f_{cb}, and for evaluating f_{cb}, effective length of compression flange, l, has to be used.

For simply supported beams and girders where no lateral restraint of the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective span of the compression flanges should be taken from Table on next page :

	Condition	Effective Span
(a)	Ends of compression flanges unrestrained against lateral bending (*i.e.*, free to rotate in plan at the bearings)	l = span
(b)	Ends of compression flanges partially restrained agaianst lateral bending (*i.e.*, not fully to rotate in plan at the bearings)	$l = 0.85 \times$ span
(c)	Ends of compression flanges fully restrained against lateral bending (*i.e.*, note free to rotate in plan at the bearings)	$l = 0.70 \times$ span

Restraint against torsion can be provided

(i) by web or flange cleats,

(ii) by bearing stiffeners acting in conjunction with the bearing of the beam,

(iii) by lateral end frames or other external supports to the ends of the compression flanges,

(iv) by walls into which the ends of the beams are built.

Where the ends of the beam are not restrained against torsion, or where the load is applied to the compression flange and both the load and the flange are free to move laterally, the values given in the table should be increased by 20 per cent.

Design of Stanchions with Battens and Lacings

The design of a stanchion generally follows the principles governing the design of compression members.

To minimise steel requirements in column design, keep the effective $\frac{l_e}{r}$ as small as possible so as to use the material at the greatest possible stress. Since $r = \sqrt{\frac{I}{A}}$

the largest radius of gyration is obtained when the material is farthest from the centroid. For constant area, this means that the material gets thinner and thinner as the column size increases for any particular type of cross-section. This leads ultimately to such thin walls for any given column cross-section that local buckling becomes a problem, and it is local buckling that ultimately limits the size to which one may go. Therefore lacing and battens are not load-carrying elements. They function primarily to load-carrying portions of the column in their relative positions and provide points of intermediate support for each separate part of the built up column.

Combined Axial Compression and Bending

Members subjected to axial compression and bending should be proportioned to satisfy the requirement:

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_m \cdot \sigma_{bcx,cal}}{\left(1 - \dfrac{\sigma_{ac,cal}}{0.60 f_{ecx}}\right)\sigma_{bcx}} - \frac{C_m \cdot \sigma_{bcx,cal}}{\left(\dfrac{1 - \sigma_{ac,cal}}{0.60 f_{ecx}}\right)\sigma_{bcx}} \leq 1.0$$

where, $\sigma_{ac,cal}$ = calculated average axial compressive stress,

σ_{ac} = a coefficient, equal to 0.85 for members in a frame where side sway is not prevented,

σ_{bc} = permissible bending compressive stress in extreme fibre,

$\sigma_{bc,cal}$ = calculated bending compressive stress in extreme fibre,

f_{cc} = elastic critical stress in compression

$= \dfrac{\pi^2 E}{\lambda^2}$

$\lambda = \dfrac{l}{r}$ = slenderness ratio in the plane of bending, #, y = x-x and y-y axes.

Design

(i) Assume some value of permissible stress σ_{ac} and calculate the approximate gross sectional area, A required. For single component sections, assume σ_{ac} = 80 MPa, and for built-up sections, σ_{ac} =110 MPa.

(ii) Select a trial section having a sectional area = approximate area calculated.

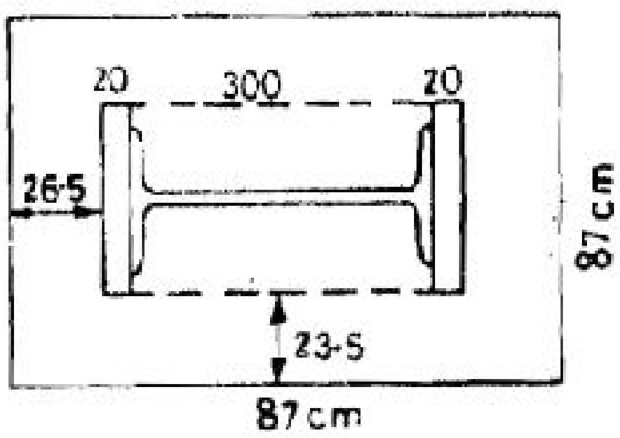
(iii) Determine the maximum $\dfrac{l}{r}$ ratio and the corresponding permissible σ_{ac}.

(iv) Calculate the safe load for the trial section. If the safe load is equal or slightly more than the design load, adopt the trial section, otherwise, revise the design.

(v) For the final design, design lacing system and battens, if required by the section configuration.

Example : A column section consisting of ISHB 300 @ 63.0 kg/m with one cover plate 400 mm wide and 20 mm thick connected to each flange carries an axial load of 300 tall inclusive. In gusseted base, allowable bearing pressure on concrete is 40 kg/cm . Allowable bending stress in base plate is 1800 kg/cm^2. Find the thickness required of the gusseted plate will be.

Ans:



Area of the base plate

$$= \frac{\text{Axial load}}{\text{Permissible stress in compression in concrete}}$$

$$= \frac{300{,}000}{40} = 7500 \text{ cm}^2$$

∴ Side of the square base plate = 86.6 cm ≈ 87 cm

∴ Adopt a slate base 87 × 87 cm.

Thickness of the base plate,

$$t = \sqrt{\frac{3w}{P_{bc}}\left(A^2 - \frac{B^2}{4}\right)} \qquad ...(i)$$

Where $w = \dfrac{300 \times 1000}{87 \times 87} = 39.6$ kg/cm

$$A = \frac{870 - (300 + 20 + 20)}{2}$$

= 265 mm = 26.5 cm.

$$B = \frac{870 - 400}{2}$$

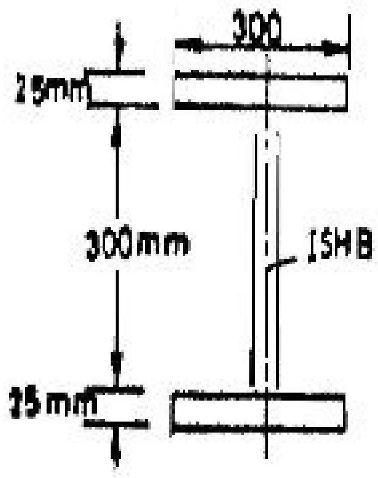
= 235 mm = 23.5 cm.

$$\therefore \quad t = \sqrt{\frac{3 \times 39.6}{1890}\left(26.5 - \frac{23.5^2}{4}\right)}$$

= 5.955 say 6 cm.

SOLVED EXAMPLE

Example: In the base plate for a column of one ISHB 300 @ 63 kg and two cover plates 300 × 25 mm as shown in the figure and carring an axial load of 230 tonnes. Indicate also the jointing to the base plate without going into the design details thereof. The column is to be supported on a concrete pedestal. The permissible bearing pressure on concrete is 40 kg/cm². It may be assumed that the column end is machined to transfer the load to the slab base by direct bearing. What is the permissible bending stress for slab base can be taken as 1890 kg/cm². The required thickness of the base plate



Ans : Area of base plate

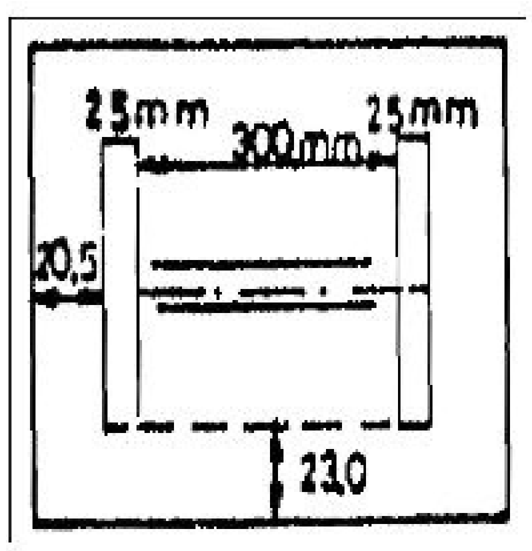
$$= \frac{\text{Axial Load}}{\text{Permissible stress in compression in concrete}}$$

$$= \frac{230,000}{40} = 5750 \text{ cm}^2$$

∴ Side of the square base plate

$$= \sqrt{5750} = 75.8, \text{ say, } 75 \text{ cm.}$$

Use a slab 76 cm × 76 cm.



Thickness of the base plate,

$$t = \sqrt{\frac{3w}{P_{bc}}\left(A^2 - \frac{B^2}{4}\right)} \quad ...(i)$$

where, $w = \dfrac{230,000}{776} = 39.82,$

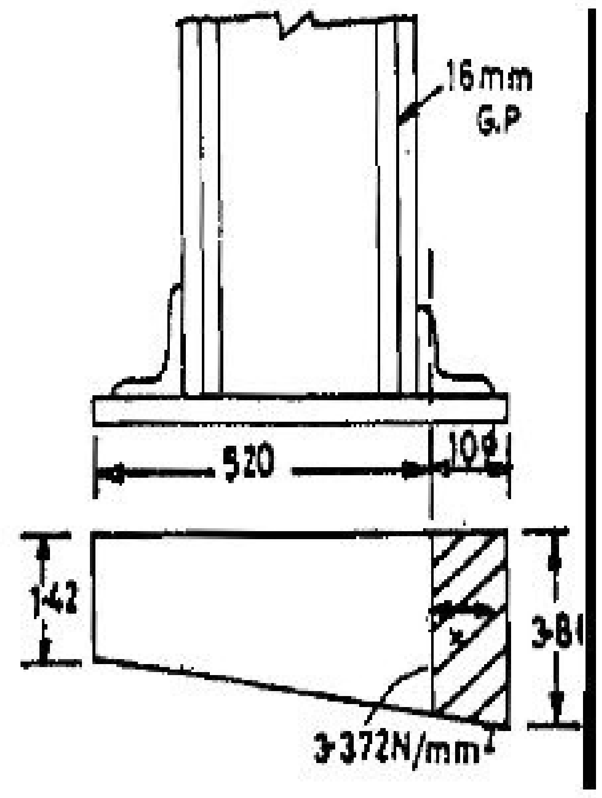
A = 23 cm, B = 20.5 cm

$$\therefore \quad t = \sqrt{\frac{3 \times 39.82}{1890}\left(23^2 - \frac{20.5}{4}\right)}$$

$= 5.135$ cm, say, 5.2 cm

Example: A column ISHB 250 carries a load 550 kAN at a distance of 40 mm from center line of column and along its *y*-*y* axis. Find size of base plate if P_{bc} = 4N/mm,

Ans.



Given, P = 550 kN, e = 40 mm

M = 22 × 10³ kN-mm

Use angle ISA 150 × 115 × 15 and 16 mm thick G.P.

$$L = 250 + 2 \times 16 + 2 \times 115 = 512$$

Take L = 520

Base Plate $\dfrac{e}{L} = \dfrac{40}{520} < \dfrac{1}{6}$

∴ Width of base plate,

$$b = \frac{P}{L \times p_{bc}}\left(1 + 6\frac{e}{L}\right)$$

$$= \frac{550 \times 10^3}{520 \times 4}\left(1 + \frac{6 \times 40}{400}\right)$$

= 386 mm.

Adopt b = 400 mm,

$$p_{bc} = \frac{550 \times 10^3}{520 \times 400}\left(1 \pm \frac{6 \times 40}{520}\right)$$

∴ p_{max} = 3.86 N/mm²,

and p_{min} = 1.42 N/mm²

$$\bar{x} = \frac{2 \times 3.86 \times 3.37}{3.86 \times 3.37} \times \frac{104}{3}$$

= 53.17 mm

$$M = \frac{3.86 \times 3.37}{2} \times 104 \times 53.17$$

= 20 × 10³ N-mm

Now $\dfrac{1}{6} t^2 \times 165 = 20 \times 10^3$

or t = 27 mm

Adopt base plate = 27 − 15 = 12 mm.

Provide 520 × 400 × 16 mm thick base plate.

DESIGN OF CONNECTIONS : COLUMN SLABS AND GUSSETED BASES

The columns transfer their loads to the soil through column bases resting over concrete or masonry blocks. A column base distributes the load over a greater area so that the pressure on the concrete block does not exceed the permissible bearing stress.

The column bases are of two types:

(*i*) Slab base, and

(*ii*) Gusseted base.

(*i*) Slab base

The column end is machined to rest over a steel base plate. Column with slab bases need not be provided with gussets, but fastenings should be provided, sufficient 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.

When the slab alone distributes the load uniformly, minimum thickness of a rectangular slab is

$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

where t = slab thickness (mm)

w = pressure or loading on the underside of the base (MPa)

σ_{bs} = permissible bending stress in slab bases = 185 MPa for steels

a = greater projection of the plate beyond the column (mm)

b = lesser projection of the plate beyond the column (mm)

When the slab does not distribute the loading uniformly or where the slab is not rectangular, special calculations are necessary to show that the stresses are within the specified limits.

(*ii*) Gusseted base.

For solid round steel columns, where the loading on the cap or under the base is uniformly distributed over the whole area including the column shaft, the minimum thickness of the square cap or base is given by

$$t = 10\sqrt{\frac{90\mathrm{W}}{16\sigma_{bs}} \times \frac{\mathrm{B}}{\mathrm{B} - d_0}}$$

where t = thickness of the plate (mm)

W = total axial load (kN)

B = length of the side of cap or base (mm)

σ_{bs} = permissible bending stress in slab bases = 185 MPa for steels

d_0 = diameter of the reduced end, if any, of the column (mm)

When the load on the cap or under the base is not uniformly distributed or where end of the column shaft is not machined with the cap or base, or where the cap or base is not square in plan, calculations should be made based on the allowable stress of 185 MPa. The base or cap should not be less than 1.5 (d_0 + 75) mm in length or diameter.

Area of slab base

Area of slab base

$$= \frac{\text{Axial load in the column}}{\text{Permissible compressive stress in concrete}}$$

Example: A rolled column section, SC 250 @ 85.6 kg/m, carries an axial load of 70 tonnes. The allowable bearing pressure on concrete is 45 kg/cm². Overall size of column section, SC 250 @ 85.6 kg/m, is 250 × 250 mm. What is the required thickness of the base plate for the column

Ans:

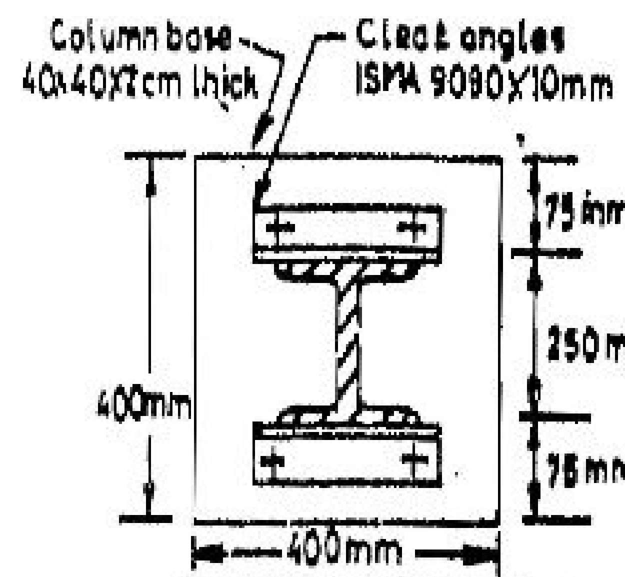


Figure Slab base for the column

Axial load of column

= 70 tonnes

Required area of slab base

$$= \frac{70 \times 1000}{45}$$

$$= 1555.56 \text{ cm}^2$$

Bearing pressure below base,

$$w = \frac{70 \times 1000}{40 \times 40}$$

$$= 43.75 \text{ kg/cm}^2$$

Thickness of the plate, $t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$

where w = 43.75 kg/cm²

$a = b$ = 75 mm = 7.5 cm

σ_{bs} = 185 MPa = 1885 kg/cm²

$$\therefore \quad t = \sqrt{\frac{3 \times 43.46}{1855}\left(7.5^2 \frac{7.5^2}{4}\right)}$$

$$= 1.71 \text{ cm} \approx 2 \text{ cm.}$$

DESIGN OF BEAMS

Flexural Members

It is a member subject primarily to bending moment. Rolled I-sections with or without cover plates are usually used for floor beams. Channel, tee and angle-sections are usually used for beams in roof trusses as purlins and common rafters.

Design Procedures

A beam section is usually selected which can resist maximum bending moment occuring over its span. The shear stress and deflection for the section are then checked to determine whether they are within the permissible limits. Check for web crippling and web buckling are the secondary design requirements.

(*i*) Design for Bending

The bending stress $\sigma_{bc \text{ or } bt}$ compressive or tensile, at any point on a cross-section of a beam due to bending moment, M is given by

$$\sigma_{bc \text{ or } bt} = \frac{M}{I} Y$$

where M = bending moment

I = moment of inertia of the cross-section

$\sigma_{bc \text{ or } bt}$ = bending stress (compressive or tensile) calculated at any point at a distance y from N.A.

The ratio $\frac{I}{Y}$ is called section modulus Z.

Since calculated bending stress $\sigma_{bc \text{ or } bt}$ should be the permissible bending stress, therefore

$$\frac{M}{Z} \leq \sigma_{bc \text{ or } bt}$$

or

$$Z \geq \frac{M}{\sigma_{bc \text{ or } bt}}$$

Suitable beam sections selected which has the sectional modulus slightly more than Z value calculated.

$$\text{Moment of Resistance} = Z \times \sigma_{bc \text{ or } bt}$$

(*ii*) Permissible Bending Stress

Laterally restrained beams : If the compression flange is restrained laterally against buckling or if the beam is to bend about the axis of minimum strength,

$$\sigma_{bc \text{ or } bt} = 0.66 fy$$

where, f_y = yield stress of steel

The permissible stress $\sigma_{bc \text{ or } bt}$ is given in the table below:

Nominal plate thickness	Yield stress, f_y (MPa)	$\sigma_{bc} = \sigma_{bt}$ (MPa)
Angle, tee, I channel and flat sections up to and including 20 mm	250	165
Over 20 mm upto and including 40 mm	240	158.4
Over 40 mm	230	151.8

(*iii*) Check for Shear

Shear stress τ at any point on the cross-section of a beam is given by

$$\tau = \frac{V}{I \times b} \times A \times \bar{y}$$

where V = shear force at the section

I = moment of inertia of the section

b = width of the section at the point where shear stress is calculated

$A \times \bar{y}$ = moment of area

For rolled I section and channel section, Average shear stress,

$$\tau_{va} = \frac{\text{shear force}}{\text{depth of beam} \times \text{web thickness}}$$

Nominal thickness (mm)	Permissible average shear stress τ_{va} (MPa)
Upto 20	100
Over 20 upto and including 40	96
Over 40	92

(*iv*) Check for deflection

Maximum deflection of a beam should not exceed $\frac{0.1}{32.5}$ of the span.

$$\text{Maximum deflection, } \delta = k \times \frac{W \times l^3}{E \times I_x} \text{cm}$$

where l = effective span

E = Young's modulus of elasticity

I_x = moment of inertia about the xx axis

K = co-efficient of maximum deflection

Example: A beam of 7 m effective span carries a uniformly distributed load of 20 N/mm including its own weight. Assume allowable stresses and deflection according to IS 800. Compression flange is held against lateral displacement. Find required section modulus

Ans. Maximum bending moment, M

$$= \frac{20 \times 7000}{8} = 1225000 \text{ N/mm}$$

$$\text{Maximum share force} = \frac{20 \times 7000^2}{2} = 70000 \text{ N}$$

Allowable bending stress, F_b = 165 N/mm^2

Allowable shear stress, F_s = 94.5 N/mm^2

Required section modulus,

$$Z = \frac{M}{f_b}$$

$$= \frac{1225 \times 10^5}{165} = 7.24 \times 10^5 \text{ mm}^3$$

DESIGN OF SEMI-RIGID AND RIGID CONNECTIONS

In the case of continuous constructions, the joints are used to transmit large moments in addition to shears. There are two kinds of construction in use for joining component members.

(*i*) **Rigid construction** in which the joints are assumed to be completely rigid so that no change of angle is permitted.

(*ii*) **Semi-rigid construction** in which deformation-moment characteristics of the joints should be known before using it. Some angle change does take place between the connected parts.

There are two alternatives :

1. The end moment is provided for by the connection of flanges with the column and the shear force is separately catered forlby a framed connection on the web of the beam.
2. A deep bracket type connection is used to resist both the end shear and moment.

Design Procedure of Moment Resistant Connection (Rigid)

Moment Connection. The moment carrying capacity of the connection is supplied by the rivets connecting the top angle (Figure *a*) or top tee (Figure *b*) with the column flange

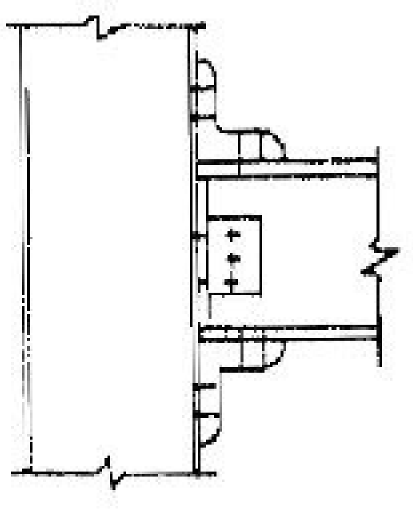


Fig. (a) Angle connection

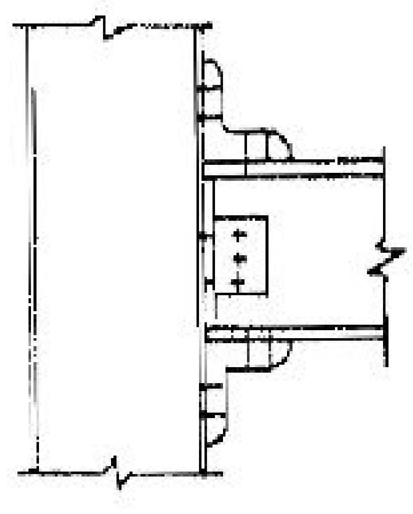


Fig. (a) Angle connection

Force in each set of rivet

$$= \frac{\text{End moment of beam}}{\text{Lever arm between top and bottom rivets}}$$

To accommodate these rivets, in lieu of T-section, many times, an I section cut longitudinally at its mid-depth into two halves, called split-beam section, is used.

$$\text{Force in the rivets (R)} = \frac{\text{Bending moment}}{\text{Lever arm}}$$

$$\text{Moment} = 0.6 \times \text{R} \times a \text{(in the case of angles)}$$

$$= 0.5 \times \text{R} \times a \text{(in the case of tee)}$$

$$\text{Thickness of the angles or tee, } t = \sqrt{\frac{\text{M} \times 6}{1890l}}$$

where, l = length of the angle of tee section.

Shear Connection

Number of rivets on web of beam and number of rivets on flange of column can be calculated from the strength of rivet.

Semi-rigid Joints

Flexible joints are those which transfer only the vertical shear and no moment. Rigid joints are those which will not allow any rotation between the connected parts. Semi-rigid connections resists the end moment, but give a relative rotation between the connected members.

Riveting and Bolting

(A) Minimum Pitch

The distance between centres of rivets should not be less than 2.5 times the nominal diameter of the rivet.

(B) Maximum Pitch

(*i*) The distance between centres of any adjacent rivets (including tacking rivets) should no exceed 300 mm or 32 *t*, whichever is less, where '*t*' is the thickness of the thinner outside plate.

(*ii*) The distance between centres of two adjacent rivets in a line lying in the direction of stress should not exceed 200 mm or 16*t*, whichever is less, in tension members, and 200 mm or 12*t*, whichever is less in compression members.

(*iii*) The distance between centres of any two consecutive rivets in a line adjacent and parallel to an edge of an outside plate should not exceed 200 mm or (100 mm + 4*t*), whichever is less in compression members or tension members.

(*iv*) When rivets are staggered at equal intervals and the gauge does not exceed 75 mm, the distance specified in (*ii*) and (*iii*) between centres of rivets may be increased by 50 per cent.

(C) Permissible Stresses in Rivets and Bolts

(*i*) **Calculation of Stresses:** In calculating shear and bending stresses the effective diameter of a rivet should be taken as the hole diameter and that of a bolt as its nominal diameter. In calculating the axial tensile stress in a rivet, the gross area should be used and in calculating the axial tensile stress in a bolt or screwed tension rod, the net area should be used.

(*ii*) **Calculation of gross and net area of rivets and bolts:** The gross area of a rivet should be taken as the cross-sectional area of the rivet hole.

(*iii*) **Area of rivet and bolt holes:** The diameter of a rivet hole should be taken as the nominal diameter of a rivet plus 1.5 mm for rivets of nominal diameter less than or equal to 25 mm, and 2.0 mm for rivets of nominal diameter exceeding 25 mm.

(*iv*) **Stresses in rivets and bolts:** The calculated stress in a mild steel shop rivet or in a bolt should not exceed the allowable values.

(*v*) Where two or more parts are connected together, a line of rivets or bolts should be provided at a distance or not more than 37 mm + $4t$ from the nearest edge, where 't' is the thickness in mm of the thinner outside plate. In the case of work not exposed to weather, this may be increased to $12t$.

WELDING

(A) Fillet Welds

(*i*) **Size of Fillet Weld:** Size of the fillet weld should not be less than the minimum allowable value given in the table.

The maximum size of the fillet weld applied to the square edge of a plate or shape should be 1.5 mm less than the nominal thickness of the edge. The size of the fillet weld used along the tee of an angle or the rounded endge of a flange should not exceed three-forth the nominal thickness of an angle of flange leg.

Table: Minimum Size of Fillet Weld

Thickness of thicker part	Minimum size
Upto and including 10 mm	3 mm
Over 10 mm upto and including 20 mm	5 mm
Over 20 mm upto and including 32 mm	6 mm
Over 32 mm upto and including 50 mm	8 mm first run
10 mm minimum	

(*ii*) **Throat of Fillet Weld :** Throat of the fillet is the length of perpendicular from the right angle corner to the hypotenuse. The effective thickness of throat is calculated as

Throat thickness = $k \times$ fillet size

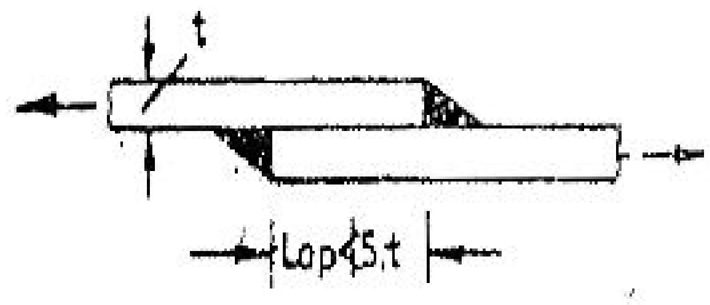
The value of K depends upon the angle between the fusion faces

In most cases, a right-angled fillet weld is used, for which

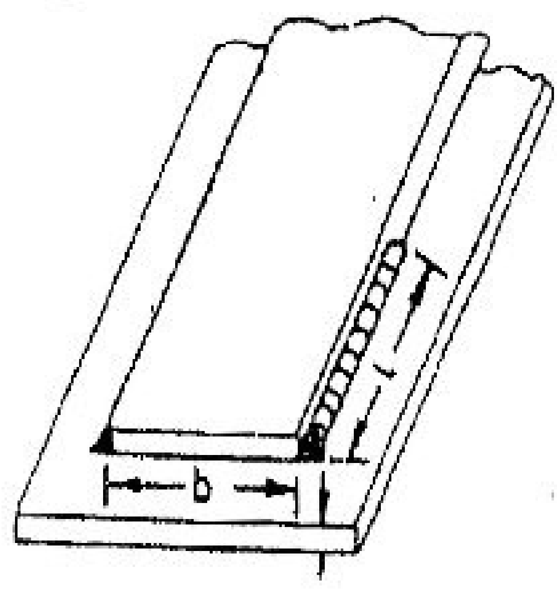
$$K = \frac{1}{\sqrt{2}} \text{ or } 0.7.$$

(*iii*) **Effective Length of Fillet Weld :** The effective length of a fillet weld is equal to its overall length minus twice the weld size. The effective length of a fillet weld designed to transmit loading should not be less than four times weld size.

(*iv*) **Overlap :** The overlap in at lap joint should not be less than five times the thickness of the thinner plate as shown in the figure.



(*v*) **Side Fillet :** In a lap joint made by a side or longitudinal fillet weld, the length of each fillet weld should not be less than the perpendicular distance between them; the perpendicular distance between the side fillets should not exceed sixteen times and thickness of the thinner part connected.



In the figure, $l < b$ and $b > 16t$,

where 't' is the thickness of thinner plate,

If 'b' exceeds the limit, the end fillet, plug weld or slot weld should be provided in addition to prevent buckling or separation of the parts.

(*vi*) **Intermittent Fillet Weld :** Any section of an intermittent fillet weld should have an effective length of not less than four times the weld size of 40 mm, whichever is greater. The clear spacing between the ends or the effective lengths of inermittent fillet weld carrying stresses should not exceed $12t$ for compression and $16t$ for tension and in no case should be more than 20 cm, here, 't' is the thickness of thinner part joined.

(*vii*) **Permissible Stress and Strength of Fillet Weld:** The permissible stress in fillet weld is 108 MPa or 1100 kgf/cm^2. The permissible stresses in shear and tension are reduced to 80% for the fillet welds made during erection.

The permissible stresses are increased by 25% if the wind or earthquake load are taken into account. However, the size of the weld should not be less than the size required when the wind or earthquake load is considered or neglected.

(B) Slot or Plug Welds

Following specifications are used for the design of slot or plug welds:

(*i*) The width or diameter of slot should not be less than three times the thickness of the part in which the slot is formed or 25 mm, whichever is greater.

(*ii*) Corners at the enclosed ends should be rounded to a radius not less than 1½ times the thickness of upper plate or 12 mm, whichever is greater.

(*iii*) The distance between the edges of the plates and the slot or between the edges of adjacent slots should not be less than twice the thickness of the upper plate.

(*iv*) The permissible stress is taken as 108 MPa or 1100 kgf/cm^2.

Design of Plate Girders

Plate girders are used when standard rolled sections do not provide the required section modulus. Building up a beam by riveting plates and angles to increase the modulus of section is one of the choices available to the designer. Plate girders are specially adopted for short spans and heavy loads.

Components : In a riveted plate girder the components are :

(*i*) web plate

(*ii*) flange plates

(*iii*) flange angles

(*iv*) stiffners (intermediate stiffners, longitudinal stiffners)

(*v*) splices

The two main components of the plate girders, namely, flanges and webs are designed to resist bending moment and shear force respectively.

PROPORTIONING OF COMPONENTS

Depth of Girders

Approximate depth of girder is usually taken as $\frac{1}{8}$ to $\frac{1}{2}$ of the effective span.

$$\text{Economical depth, D} = 5.5\sqrt[3]{\frac{m}{f_b}}$$

Width of Flange : Width of flange would be within the range of

$$\frac{L}{40} \text{ to } \frac{L}{45}$$

Deflection : The maximum deflection should not exceed $\frac{1}{325}$ of the span.

Allowable Stresses

Upto 20 mm web thickness

Bending stress = 1575 kg/cm^2,

Shear stress = 945 kg/cm^2

Over 20 mm thick web

Bending stress = 1500 kg/cm^2

Shear stress = 865 kg/cm^2

Minimum Thickness of Plates

For those girders exposed to weather but accessible for painting = 6 mm

Not accessible for painting = 8 mm

Analysis : Moment of resistance of plate girders can be obtained by

1. The flange area method, and
2. The moment of inertia method

Plate girder is a built-up beam and moment of resistance can be found out by flexure formula.

$$M = \frac{fI}{y}$$

where M = moment of resistance

I = moment of inertia

f = maximum allowable stress, and

y = extreme fibre distance

Flange Area Method

This method is approximate but is sufficiently accurate for practical purposes.

Moment of girder section, $M = F \times D$ ...(*i*)

where, F = Resultant compressive and tensile force

Average bending stress $f_b = A$ ...(*ii*)

where, A = flange area

D = Distance between lines of action of these forces

Combining equations (*i*) and (*ii*), we get

$$M = f_b AD$$

Resisting moment of the flange,

$$M_F = A_g \times f_b \times D$$

where, A_g = gross area of one flange

D = distance between centroids of flanges.