STEEL STRUCTURES

TYPE OF SECTION

Rolled Steel Sections

Steel structures are built with hot-rolled steel sections. The Indian Standards Institution has evolved a rational, efficient and economical series of Indian Standards (IS: 808-1964 and its parts, part I—1973, part II—1978. part III— 1979. part V—1976, and part VI—1976) for rolled steel beams, channels and angle sections to save steel in construction works. The following sections are standardized by the Indian Standards Institution.

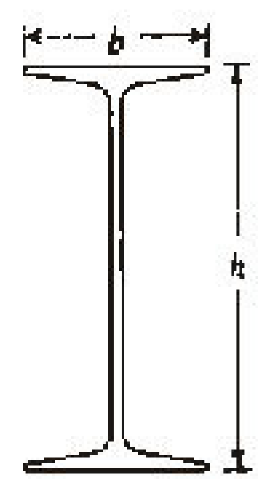


Fig: Beam Section

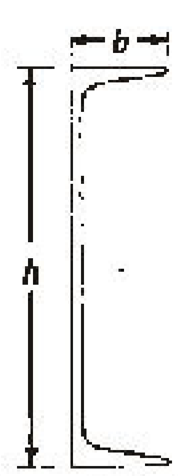


Fig: Channel Section

(i) I-Sections

(a) Indian Standard Junior Beams (ISJB).
(b) Indian Standard Light Beams (ISLB).
(c) Indian Standard Medium Weight Beams (MB).
(d) Indian Standard Wide Flange Beams (ISWB).
(e) Indian Standard Column Section (SC) All above I-sections are designated along with the depth the respective section in mm, e.g. MB 200 is a hot-rolled steel, medium-weight beam of depth 200 mm.

(ii) Channel-sections

(a) Indian Standard Gate Channel (ISPG).
(b) Indian Standard Junior Channel (ISJC).
(c) Indian Standard Light Channel (ISLC).
(d) Indian Standard Medium Weight Channel with sloping flange (MC).
(e) Indian Standard Medium Weight Channel with parallel flange (MCP).

All the above hot-rolled channel sections are designated along with the depth of the respective section, e.g. MC 200 is a medium weight channel of depth 200 mm.

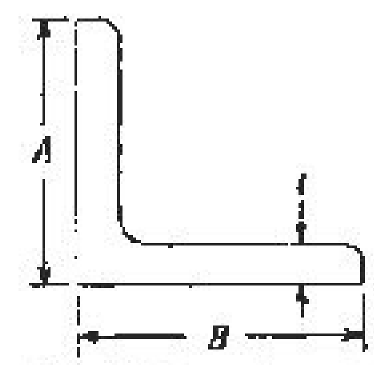


fig. (a) Equal angle Section A=B

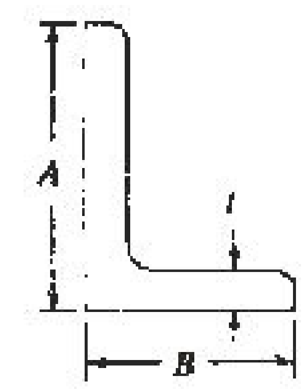


fig. (b) Unequal angle Section A≠B

(iii) Angle-sections

(a) Indian Standard Equal Angles.
(b) Indian Standard Unequal Angles.

Angle-sections are designated by abbreviation ISA along with the lengths of both legs and their thickness, e.g., ISA 6565, 8 mm or ISA 65 x 65 x 8 mm is an equal-angle section 8 mm thick and with both legs 65 mm long. The supplementary angle sections are designated by the size of "legs and their thickness without the prefix ISA.

(iv) Tee-sections

(a) Indian Standard Rolled Normal Tee Bars (ISNT).
(b) Indian Standard Rolled Deep Legged Tee Bars (ISDT).
(c) Indian Standard Slit Light Weight Tee Bars (ISLT).
(d) Indian Standard Slit Medium Weight Tee Bars (ISMT).
(e) Indian Standard Slit Tee Bars from H-sections* (ISHT).

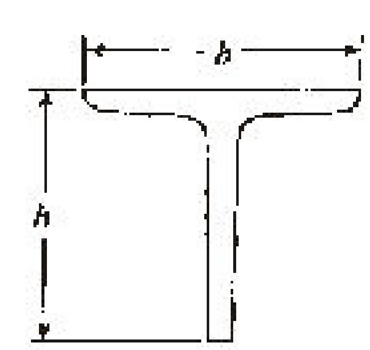
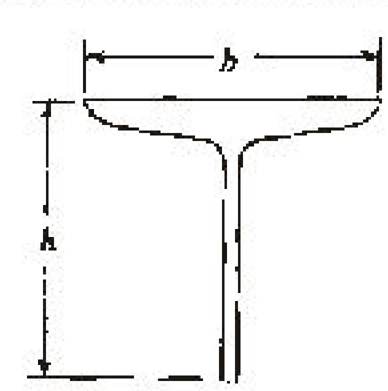


Fig. 1.14 (a) Rolled Normal Tee Bar



(b) Slit Tee Bar and Deep Legged Tee Bar

Tee-sections are designated by the respective abbreviations followed by their depth, e.g. a normal tee-bar of depth 100 mm is designated by ISNT 100.

TYPES OF STRUCTURAL STEELS

Table 1.1 gives the various types of structural steels and their strengths.

SPECIFICATIONS

This is based upon IS: 800—1984 (second revision), Indian Standard code of practice for general steel

Table

Type of steel	Class of production	Nominal thickness (mm)	Tensile strength (N/mm^2)	Yield stress (N/mm^2)
RS :226/75				
(Standard	Plates, sections,	Up to 20	410 to 530	250
quanlity)	angles, tees,	>20 to 40	410 to 530	240
	beams, channels.	Over 40	410 to 530	230
	etc., flats.			
	Bars — round,	Up to 20	410 to 530	250
	square and	Over 20	410 to 530	240
	hexagonal			
RS: 961/75				
(High tensile)	Plates, sections,	Up to 28	570	350
St 58 HT	angles, beams,	> 28 to 45	570	340
	channels, etc..	> 45 to 63	570	320
	bars, flats	Over 63	540	290
SI 55 HTW	Plates, sections.	Up to 16	540	350
	bars, flats	> 16 to 32	540	340
		> 12 to 63	510	330
		Over 63	490	280
RS: 2062/84				
(Fusion welding	Plates, sections —	Up to 20	410	250
quality)	angles, beams, tees,	> 20 to 40	410	240
	etc., flats	Over 40	410	230
RS. 1977/75				
(Ordinary	Plates, sections —	Up to 20	410-530	250
quality)	angles, beams, etc..	>20 to 40	410-530	240
Fe 410-0	flats.	Over 40	10-530	230
	Bars	Up to 20	10-530	250
		Over 20	10-530	240
RS: 8500/77				
Fe 440 HTI	Plates, sections	< 6	440 to 560	300
and	angles, beams.	>6 to 20	440 to 560	300
Fe 440 HT2	channels, etc.,	> 20 to 40	440 to 560	290
	bars, fiats	> 40 to 63	440 to 560	280
Fe 540 HT.	Plates, sections	< 6	540 to 660	410
540 HTA	angles, beams.	> 6 to 20	540 to 660	400
and	channels, etc.,	> 20 to 40	540 to 660	390
Fe 540 HTB	bars, flats	> 40 to 63	540 to 660	380
Fe 570 HT	-do-	< 6	570 to 720	450
		>6 to 20	570 to 720	440
		> 20 to 40	570 to 720	430
		> 40 to 63	570 to 720	420
Fe 590 HT	-do-	< 6	590 to 740	490
		> 6 to 20	590 to 740	480
Fe 640 HT	-do-	< 6	640 to 790	540
		> 6 to 20	640 to 790	530

construction. Section 2 of IS: 800—1984 may be referred for various other relevant codes. The various loads on the structures are estimated as per the IS: 875 code of practice for the structural safety of buildings.

CONNECTIONS

RIVETING

The size of the rivet is the diameter of the shank.

(a) Gross dia of rivet or dia of hole

$\boxed{d' = d + 1.5mm}$ for $d \leq 25$ mm

and $\boxed{d' = d + 2.0mm}$ for $d > 25$ mm

where, d = Nominal dia of rivet

d' = Gross dia of rivet or dia of hole.

Note : For strength calculation effective diameter is taken into account. This is based on the assumption that rivet fills the hole completely.

(b) Unwins formula

$$\boxed{d_{mm} = 6.05\sqrt{t_{mm}}}$$

Where,

d_{mm} = dia of rivet in mm

t_{mm} = thickness of plate in mm.

BOLTED JOINTS

Bolts may be used in place of rivets for structure not subjected to vibrations. The following types of bolts are used in structures:

(i) Black bolts :

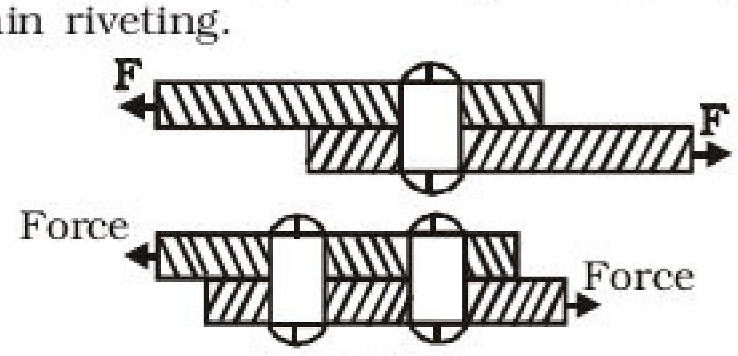
- Hexagonal black bolts are commonly used in steel works.
- They are made from low or medium carbon steels
- They are designated as black bolts $M \times d \times l$
  Where, d = diameter, and l = length of the bolts.

(ii) Precision and Semi Precision Bolts :

- They are also known as close tolerance bolts.
- Sometimes to prevent excessive slip, close tolerance bolts are provided in holes of 0.15 to 0.2 mm oversize. This may cause difficulty in alignment and delay in the progress of work.
- **Types of Riveted and Bolted Joints**
  There are two types of riveted or bolted joints.

(i) Lap joint :

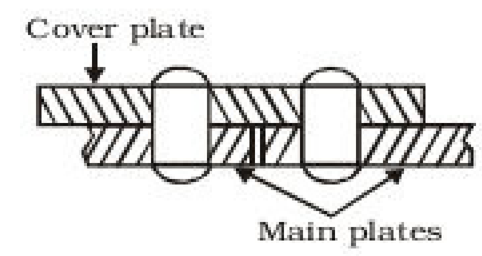
- The lap joint is that in which the plates to be connected overlap each other.
- The lap joint may have single-row, staggered or chain riveting.



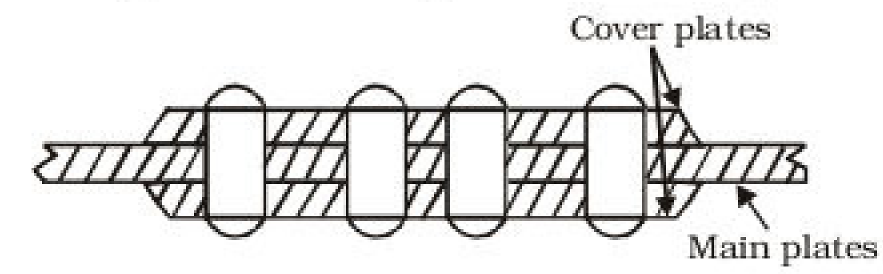
Lap joint

(ii) Butt Joint :

- The butt joint is that in which the plates to be connected butt against each other and the connection is made by providing a cover plate on one or both sides of joint.
- The butt joint may have a single row or staggered or chain riveting.



Single-riveted single-cover butt joint



(i) Nominal diameter (d) : The diameter of the shank of a rivet before riveting, is called the nominal diameter. For a bolt, the diameter of the unthreaded portion of the shank is called its nominal diameter.

(ii) Effective diameter or gross diameter: The effective or gross diameter of a rivet is equal to the diameter of the hole it fills after riveting. For a bolt, the nominal diameter is same as the gross diameter.

(iii) Net area: The net area of a bolt is the area at the root of the thread.

(iv) Gauge: A row of rivets parallel to the direction of force is called a gauge line. The normal distance between two adjacent gauge line is called the gauge.

(v) Edge distance: It is the distance between the edge of a member or cover plate and the centre of the nearest rivet hole.

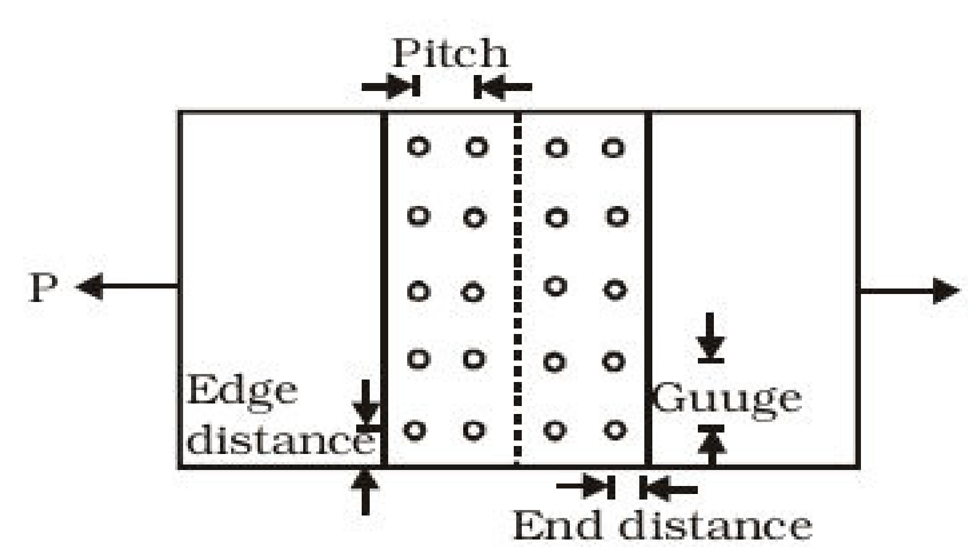
(vi) Proof load: Initial tension in HSFG bolts is known as proof load of the bolt.

(vii) Slip Factor: Coefficient of friction in friction type joint is known as slip factor.

(viii) Pitch : The distance between centres of any two adjacent rivets parallel to the direction of force is called pitch. Diagonal pitch is the distance between centres of any two adjacent rivets in the diagonal direction is called diagonal pitch.

FAILURE OF RIVETED/BOLTED JOINTS

(i) By Tearing of plate between rivets



Strength of tearing per pitch length

$$\boxed{P_t = (P - d')t \times f_t}$$

Where, f_t = Permissible tensile stress in plates

t = Thickness of plate

d' = Dia of hole (gross dia of rivet)
P = Pitch

(ii) Strength of rivet in single shear

$$\boxed{P_s = \frac{\pi}{4}(d')^2 . f_s}$$

(iii) Strength of rivet in double shear

$$\boxed{P_s = 2 \times \frac{\pi}{4} d'^2 . f_s}$$

where, f_s = allowable shear stress in rivets
d' = dia of hole.

(iv) Failure due to bearing or crushing of rivet or plates

Strength of rivet in bearing

$\boxed{P_b = f_b . d' . t}$ where, f_b = bearing strength of rivet.

Note : Shearing strength of joint is simply the sum of shearing strength of individual rivets. Bearing strength of joint is simply sum of bearing strength of individual rivets in the joints.

EFFICIENCY OF JOINTS (η) :

$$\boxed{\eta = \frac{Minimum\{P_s, P_b, P_t\}}{P}}$$

where,
P_s = Strength of joint in shear
P_b = Strength of joint in bearing
P_t = Strength of joint in tearing
P = Strength of plate in tearing when no deduction has been made from rivet holes = $p.t.f_t$

- Rivet value $\boxed{R_V = \text{minimum}\begin{cases} P_s \\ P_b \end{cases}}$
- Number of rivet, $\boxed{n = \frac{Force}{R_v}}$
- ***I.S.* 800; 1984 Recommendation**
  Maximum permissible stress in rivets & bolts

Type of fastener	Axial tension, s_{at} (MPa)	Shear, t_{vf} (MPa)	Bearing, s_{pf} (MPa)
(i) Power driven			
(a) Shop rivets	100	100	300
(b) Field rivets	90	90	270
(ii) Hand driven rivets	80	80	250
(iii) Close tolerance and turned bolts	120	100	300
(iv) Bolts in clearance holes	120	80	250

- Rivet diameter, Pitch

Minimum pitch	2.5 times of nominal diameter of the rivet
Maximum pitch for	
(i) any two adjacent rivets (including tacking rivets)	32t or 300mm, whichever is less
(ii) rivets lying in a line parallel to the force in the member:	
(a) in tension	16t or 200mm, whichever is less
(b) in compression	12t or 200mm, whichever is less

where, t = thickness of thinner outside plate

PERMISSIBLE STRESSES

Case	Permissible stress
Axial tension and compression	$0.60f_y$
In bending	$0.66f_y$
In bearing(ex at support)	$0.75f_y$
In shear	max. permissible avg. $0.40f_y$ max. permissible $0.45f_y$

MAX PERMISSIBLE DEFLECTIONS

(a) Max permissible horizontal and vertical deflection = $\frac{\text{Span}}{325}(WSM)$

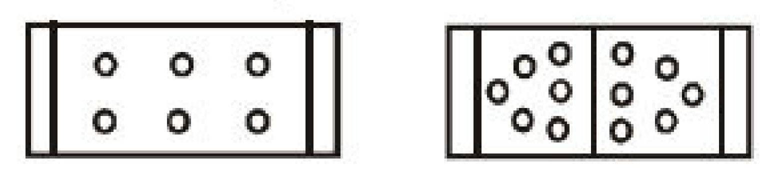
(b) Max permissible deflection when supported elements are susseptible to cracking
$= \frac{\text{Span}}{360}(LSM)$

(c) Max permissible deflection when supported element are not susseptible to cracking
$= \frac{\text{Span}}{300}(LSM)$

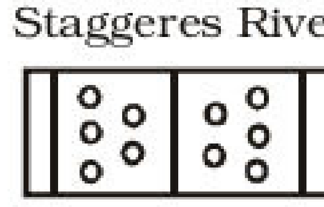
1. When wind and earthquake loads are considered permissible stresses in steel structures are increased by 33.33% and in rivets and welds are increased by 25%.
2. By providing proper edge distance, we can prevent shear failure, splitting failure and bearing failure of plates.

ARRANGEMENT OF RIVETS

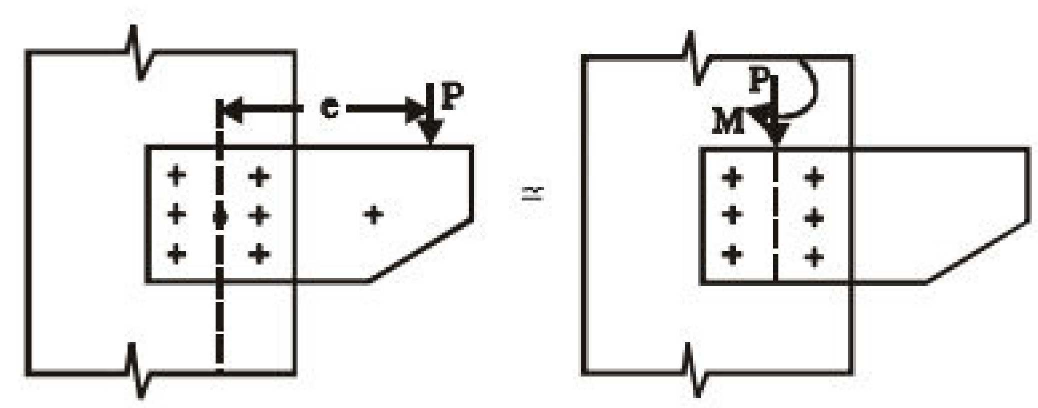
(a) Chain Riveting (b) Diamond Riveting



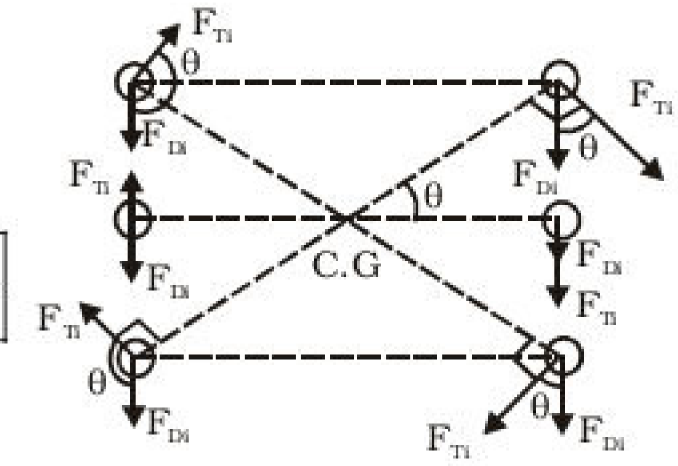
(c) Staggeres Riveting



ECCENTRIC CONNECTIONS



(i) $\boxed{F_{Di} = \frac{P.A_i}{\Sigma A_i}}$



(ii) $\boxed{F_{Ti} = \dfrac{Per_i}{\Sigma A_i r_i^2} A_i}$

Special Case: When all the rivets are of same diameter then.

(a) $\boxed{F_{Di} = \dfrac{P}{n}}$ (b) $\boxed{F_{Ti} = \dfrac{Per_i}{\Sigma r^2}}$ or $\boxed{F_{Ti} = \dfrac{Mr_i}{\Sigma r^2}}$

(iii) $\boxed{Fr_i = \sqrt{(F_{Di})^2 + (F_{Ti})^2 + 2F_{D_i}F_T \cos\theta} \le R_v}$

where, F_{Di} = Direct force in i^{th} rivet.
F_{Ti} = Force in i^{th} rivet drew to torsional moment
r_i = Distance of i^{th} rivet from $C.G.$

A_i = Area of i^{th} rivet = $\dfrac{\pi}{4}(d_i)^2$

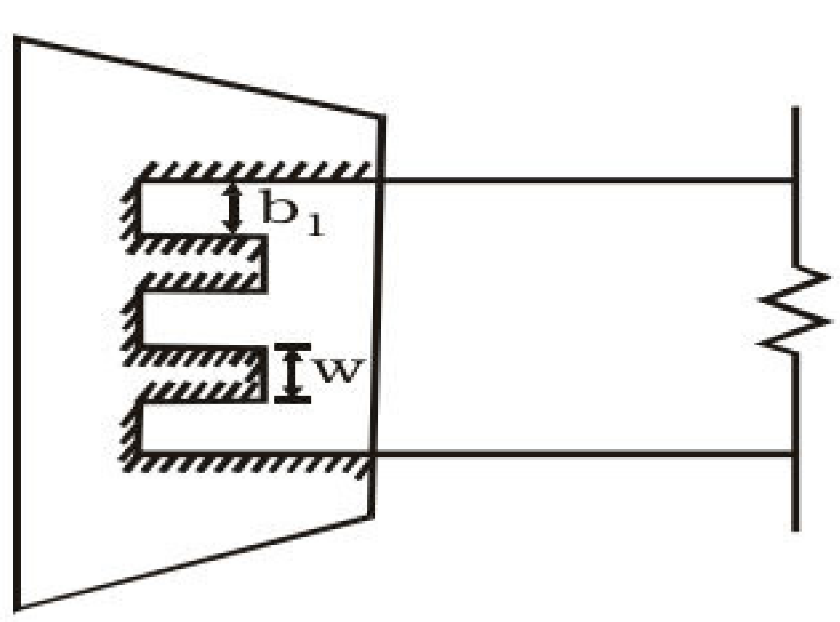
F_{Di} = Always acts in the direction of applied load P.
F_{Ti} = Always acts perpendicular to the line joining $C.G.$ of rivet group and the rivet under consideration.
F_{ri} = Resultant force in i^{th} rivet.
Note: Most critical rivet is one for which θ is minimum and r is maximum.

Angle between fusion faces	Value of K
60 –90	0.70
91 –100	0.65
101 106	0.60
107 113	0.55
114 120	0.50

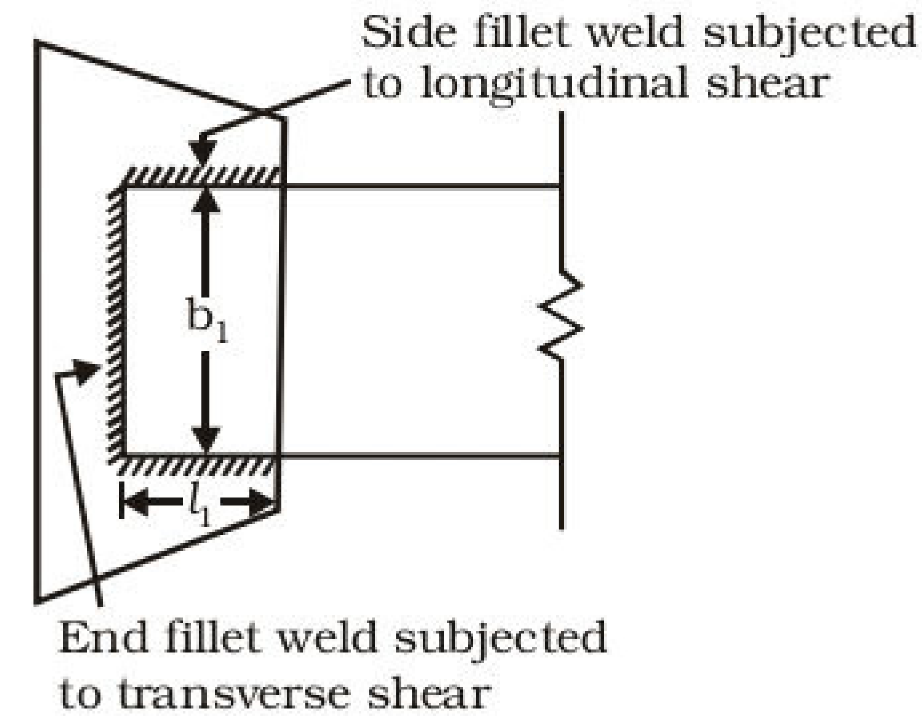
- **Minimum size of weld**
It depends upon thickness of thicker plate

Thickness of thicker plate	Minimum size
0 10 mm	3 mm
11 20 mm	5 mm
21 32 mm	6 mm
32 mm	8 mm

- **Slot weld** For welds in compression zone max clear spacing between effective
length of weld = $12t$ or 200 mm (minimum).
In tension zone = $16t$ or 200 mm (minimum)
- **Slot weld**



- **Side fillet weld**



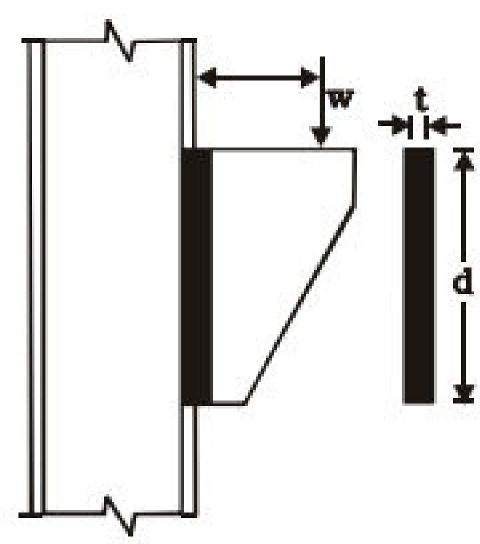
(a) $l_1 \not< b_1$
(b) $b_1 \not> 16t$ to make stress distribution uniform
(c) if $b_1 > 16t$ use end fillet weld.

WELDED CONNECTION

- **Permissible Stresses**

(a) Tensions and compression on section through the throat of butt weld = 150 N/mm^2
(b) Shear on section through the throat of butt of fillet weld = 108 $N/mm^2 \cong 110\ N/mm^2$
Throat thickness $t = k \times$ size of weld

- **Butt-welded joint Loaded Eccentrically**
  - Let thickness of weld throat = t, and length of weld = d



- Shear stress at weld, $\boxed{P_s = \dfrac{W}{d \times t}}$

where, t = thickness of weld throat and d = length of weld.

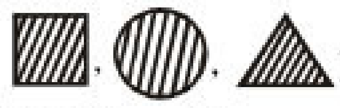
- Tensile or compressive stress due to bending at extreme fibre, $\boxed{P_b = \dfrac{6M}{t \times d^2}}$ For the safety of joint the ineraction equation.

$$\left[\left(\frac{P_s}{\text{Permissible shear stress in weld}}\right)^2 + \left(\frac{P_b}{\text{Permissible tensile stress in weld}}\right)^2\right] \le 1$$

- **Equivalency Method**

$\sqrt{P_b^2 + (3P_s)^2} \le 0.9 f_y$ (based on max distortion energy theory)

Permissible bending stress for flanged section $= 165\ N/mm^2 = 0.67 f_y$

For solid section  permissible bending stress is 185 N/mm^2

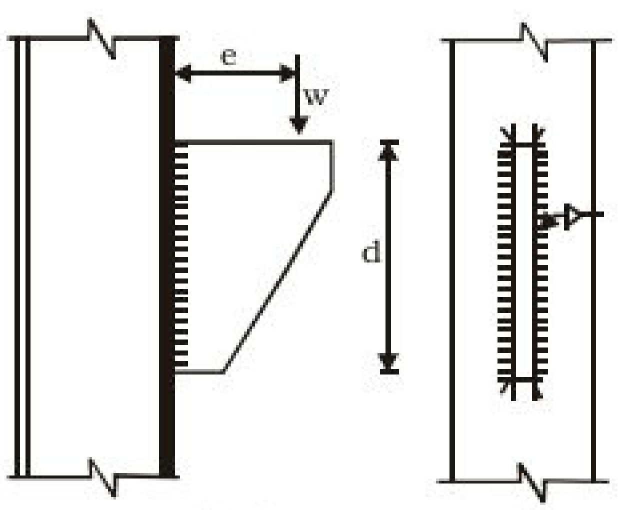
FILLET-WELDED JOINT LOADED ECCENTRICALLY

There can be two cases:

(i) Load not lying in the plane of the weld
(ii) Load lying in the plane of the weld

(i) Load not lying in the plane of the we :

- Let thickness of weld throat = t and total length of weld = $2 \times d$
- Vertical shear stress at weld, $\boxed{p_s = \frac{W}{2d \times t}}$



- Horizontal shear stress due to bending at extremefibre, $\boxed{p_b = \frac{M}{I} \times y = \frac{(W \times e) \times d/2}{\frac{2 \times t \times d^3}{12}}}$

$$\boxed{p_b = \frac{3We}{td^2}}$$

- Resultant stress, $\boxed{p_r = \sqrt{p_s^2 + p_b^2}}$
- The value of p_r should not exceed the permissible shear stress p_q (= 108 MPa) in the weld.
- For design of this connection, the depth of weld may be estimated approximately by

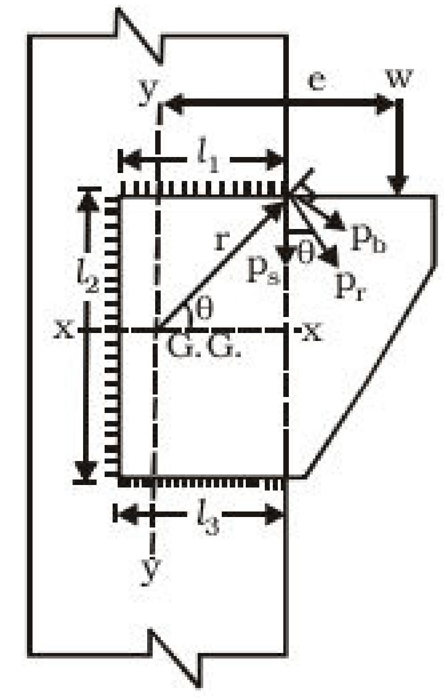
$$\boxed{d = \sqrt{\frac{6 \times W \times e}{2 \times t \times p_b}}}$$

(ii) Load lying in the plane of the weld: Consider a bracket connection to the flange of a column by a fillet weld as shown in figure.

- Vertical shear stress at weld, $\boxed{p_s = \frac{W}{l \times t}}$

where,
$l(l_1 + l_2 + l_3)$ = the lenght of weld
and t = thickness of the throat

- Torsional stress due to moment, at any point in the weld, $\boxed{p_b = \frac{T \times r}{l_p}}$



where,
T = torsional moment = $W \times e$
r = distance of the point from centroid of weld section
l_p = polar moment of inertia of the weld group $= l_x + l_y$

- The resultant stress, $\boxed{p_r = \sqrt{p_s^2 + p_b^2 + 2p_s p_b \cos\theta}}$
- For safety, $p_r \not> $ permissible stress in fillet weld, *i.e.* 108 MPa.
- The resultant stress p_r will be maximum at a point where r is maximum and q is minimum.

TENSION MEMBER

1. Tension member has no stability problem.
2. In tension member net section will be effective whereas in compression member gross section is effective.

Type of member	Max. Slenderness Ratio
1. A tension member in which reversal of direct stress due to loads other then wind or earthquake forces	180
2. A member normally acting as a tie in roof truss or bracing system. But subjected to possible reversal of stress resulting from the action of wind or earthquake forces.	350

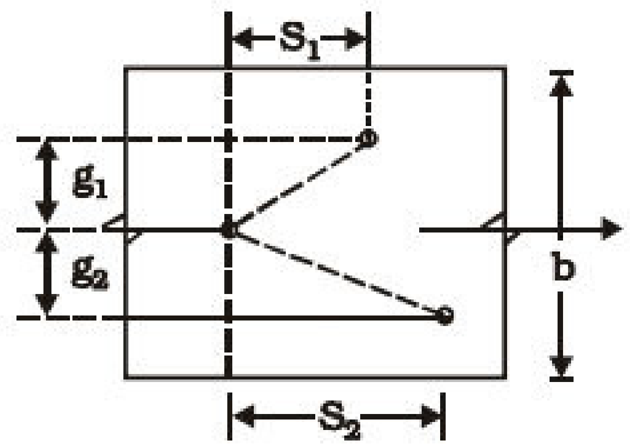
NET SECTIONAL AREA

(i) For Plate

$$\boxed{Net\ Area = (b \times t) - nd' t + \left(\frac{s_1^2}{4g_1} + \frac{s_2^2}{4g_2}\right) t}$$

where,

S_1 = Distance between two consecutive rivets in the direction of load, also called pitch.
g_1 = Distance between two consecutive rivets perpendicular to the direction of load also called gauge.
b = Width of the plate
n = Number rivets at the section
t = Thickness of the plate
d' = Gross diameter of the rivet

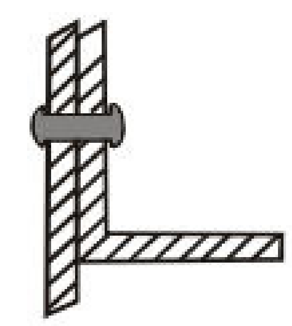


(ii) Single angle connected by one leg only.

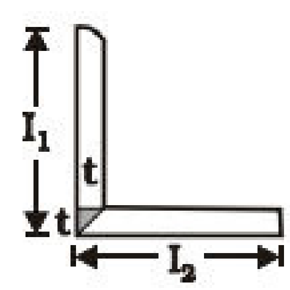
(a) $\boxed{A_{net} = A_1 + kA_2}$

where, A_1 = Net cross-section area of the connected leg.

A_2 = Gross cross-sectional area of unconnected leg. (outstand)



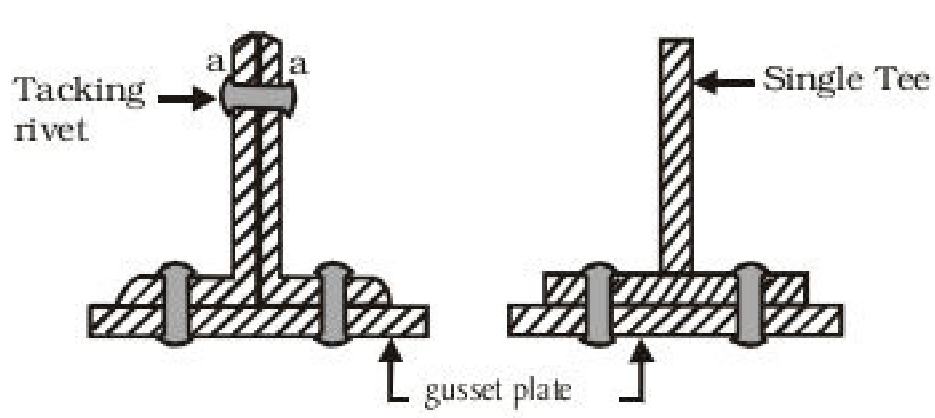
(b) $\boxed{k = \dfrac{3A}{3A_1 + A_2}}$



(C) $\boxed{A_1 = \left(l_1 - \dfrac{t}{2}\right)t}$ (d) $\boxed{A_2 = \left(l_2 - \dfrac{t}{2}\right)t}$

(e) $\boxed{A_{net} = (l_1 + l_2 - t)t}$

(iii) For pair of angle placed back to back (or a signal tee) connected by only one leg of each angle (or by the flange of a tee) to the same side of a gusset plate : or It the two angles are tagged along a-a.



(a) $\boxed{A_{net} = A_1 + kA_2}$

(b) $\boxed{k = \dfrac{5A_1}{5A_1 + A_2}}$

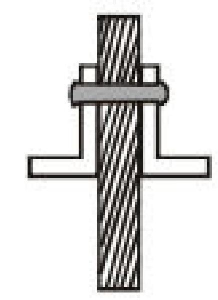
where, A_1 = Area of connected leg

A_2 = Area of outstand (unconnected leg)

(c) The area of a web of tee = Thickness of web × (depth – thickness of flange)

(d) The outstand legs of the pair of angles should be tacked by rivets of a pitch not exceeding 1 m.

(iv) If two angles are placed back to back and connected to both sides of the gusset plate. Then



$A_{net} = A_1 + A_2$ (k = 1) when tack riveted.
If not tack riveted then both will be considered separately and case (ii) will be followed $k = \dfrac{3A_1}{3A_1 + A_2}$

PERMISSIBLE STRESS IN DESIGN

- The direct stress in axial tension on the effective net area should not exceed σ_{at}
  where, $\sigma_{at} = 0.6 f_y$
  and f_y = minimum yield stress of steel in MPa
- Allowable stress σ_{at} in Axial Tension for steel Conforming to **IS : 226-1975.**

Form	*Thickness / Diameter*	*$_{at}$(MPa)*
1. *Plates, angles, tees and I beams channels and flats*	*Upto and including 20mm*	150
	20mm to 40mm	144
	Over 40mm	138
2. *Bars (round, square) and hexagonal*	*UP and including 20mm*	150
	Over 20mm	144

LUG ANGLE

The lug angle is a short length of an angle section used at a joint to connect the outstanding leg of a member, thereby reducing the length of the joint. When lug angle is used k = 1.

COMPRESSION MEMBER

STRENGTH OF AN AXIALLY LOADED COMPRESSION MEMBER

- The maximum axial compressive load P

$\boxed{P = \sigma_{ac} \times A}$

where, P = axial compressive load (N)

σ_{ac} = permissible stress in axial compression (MPa).

A = gross-sectional area of the member (mm^2).

- IS : 800-1984 uses the Merchant Rankine formula for σ_{ac} which is given as

$$\boxed{\sigma_{ac} = 0.6 \times \frac{f_{cc} \times f_y}{\left[f_{cc}^n + f_y^n\right]^{1/n}}}$$

where, f_{cc} = elastic critical stress in compression

$$= \frac{{}^{2}\quad E}{{}^{2}}$$

λ = slenderness ratio = $\frac{l}{r}$

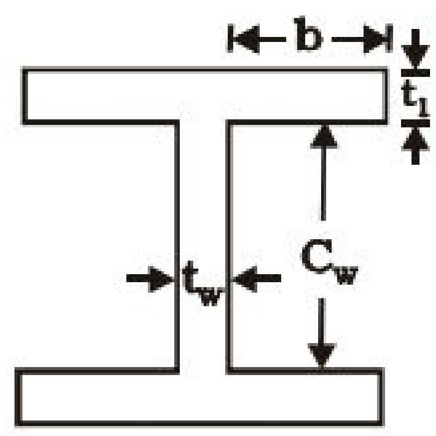
l = effective length of the compression member.

r = appropriate radius of gyration of the member (minimum value).

E = modulus of elasticity of steel = 2×10^5 MPa

n = a factor assumed as 1.4

- According to IS : 800-1984, the direct stress in compression on the gross cross-sectional area of axially loaded compression member shall not exceed $0.6f_y$, nor the permissible σ_{ac} value calculated using the above formula.
- The critical stress at which the plate buckles is inversely propational to $(b/t)^2$.
- To prevent the buckling of flange plate and web plate.



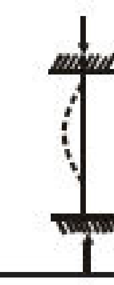
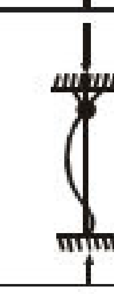
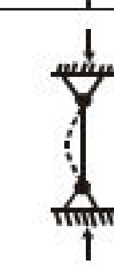
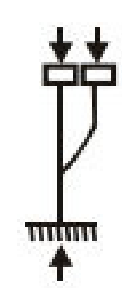
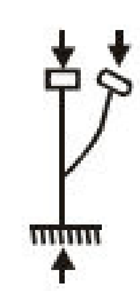
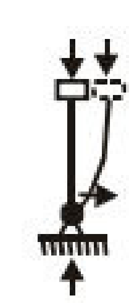
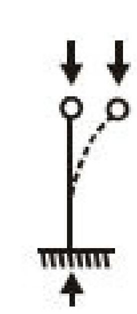
$$\frac{b}{t_1} \ngtr 16, \ \frac{d_w}{t_w} \ngtr 50$$

- Maximum Slenderness Ratio (Clause 3.7.1 IS:800-1984)

S.N.	Type of memebr	Max. slenderness ratio
1.	A member carrying compressive loads resulting from dead load and superimposed loads	180
2.	A member subjected to compressive loads resulting from wind/earthquake forces provided the deformation of such members does not adversely affect the stress in any part of the structure	250
3.	A member normally carrying tension but subjected to reversal of stress due to wind or earthquake forces	350

EFFECTIVE LENGTH

Table : (Effective length of compression members of compression members of constant dimensions (Clause 5.2.2 IS : 800-1984)

S.N	Degree of end restraint of compression	Recommended value of effective Length member	Symbol
1.	Effectively held in position and restrained against rotation at both ends	0.65L	
2.	Effectively held in position at both ends restrained against rotation at one end	0.80L	
3.	Effectively held in position at both ends, but not restrained against rotation	1.00L	
4.	Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position.	1.20L	
5.	Effectively held in position and restrained against rotation at one end, and at the other end partially restrained against rotation but not held in position	1.50L	
6.	Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position	2.00L	
7.	Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end	2.00L	

Angle Struts (Clauses 5.5, IS : 800-1984)					
S.N	**Type**	**End Connections**	**Effective length**	**Allowable stress**	**Slenderness Ratio**
1.	Single angle discontinuous	(i) One rivet or bolt at each end (ii) Two or more rivets or bolts or welding at each end	$l = L$ $l = 0.85L$	$0.8\sigma_{ac}$ σ_{ac}	180 -
2.	Double angle, tacked, discontinuous	(i) Connected on same side of gusset plate (a) Onerivet or bolt at each end (b) Two or more rivets, bolts or welding at eachend (ii) Connected on both Sides of gusset plate by two or more rivets, bolts or welding	$l = L$ $l = 0.85L$ $l = 0.7$ to $0.85L$ depending on rigidity of joint	$0.8\sigma_{ac}$ σ_{ac} σ_{ac}	$\frac{l}{r} \ngtr 180$ - -
3.	Single or double angle continuous	One or more rivet, bolt or welding	$l = 0.7L$ to $1.0L$ depending on end rigidity	σ_{ac}	-

For battened columns, the effective length shall be increased by 10%.

ANGLE STRUTS

- The slenderness ratio ($\lambda = l/r$) should not exceed the values given in Tabel).

BUILT-UP COMPRESSION MEMBER

Tacking Rivets

- The slenderness ratio of each member between the connections should not be greater than 40 nor greater than 0.6 times the most unfavorable slenderness ratio of the whole strut. In no case should the spacing of tacking rivets in a line exceed 600 mm for such members *i.e.* two angles, channels or tees placed back-to-back.
- For other types of built-up compression members, say where cover-plates are used, the pitch of tacking rivets should not exceed 32 t or 300 mm, whichever is less, where t is the thickness of the thinner outside plate. When plates are exposed to the weather, the pitch should not exceed 16 t or 200 mm whichever is less.
- The diameter of the connecting rivets should not be less than the minimum diameter given below.

Thickness of member	Minimum diameter of rivets
Up to 10 mm	16 mm
Over 10 mm to 16 mm	20 mm
Over 10 mm	22 mm

DESIGN OF COMPRESSION MEMBERS

The following steps are followed for designing an axially loaded compression member:

(i) Assume some value of permissible compressive stress σ_{ac} and calculate the approximate gross sectional area A required.

$$A_{approx} = \frac{\text{Axial compressive load}}{\text{Assumed permissible stress}}$$

For single-angle-channel-or I-section (low loads) 80 MPa and for built-up sections (heavy loads) 110 MPa may be assumed initially as permissible compressive stress.

(ii) Choose a trial section having area $\simeq A_{approx}$.

(iii) Determine the actual permissible stress corresponding to maximum slenderness ratio l/r of the trial section.

(iv) Calculate the safe load to be carried by trial section by multiplying, the actual permissible stress by the area of the trial section.

If the safe load is equal to or slightly more than the actual load, the trial section is suitable for selection. Otherwise the above steps should be repeated.

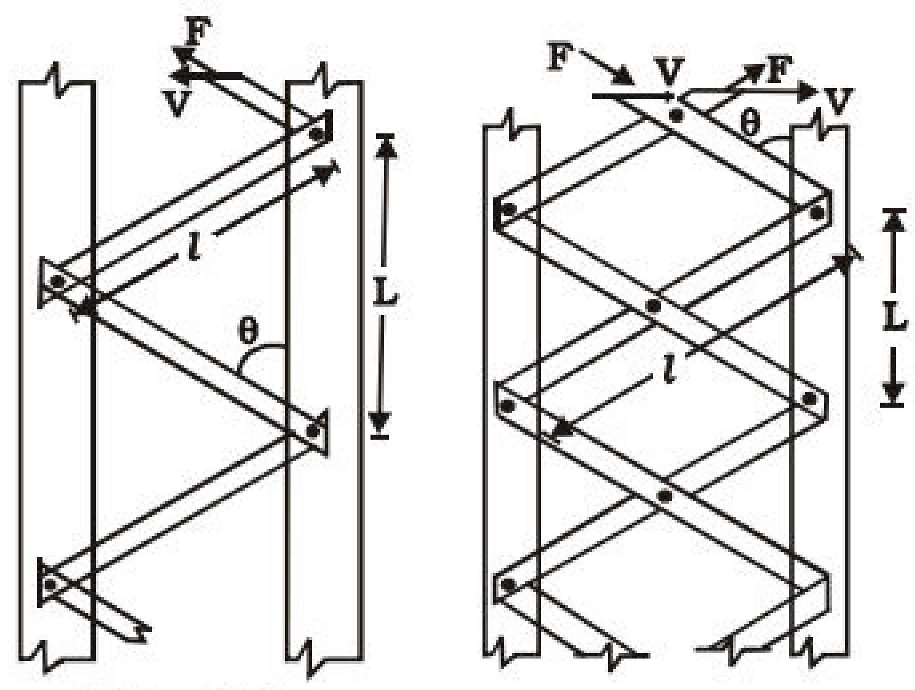
(v) Check the slenderness ratio.

LACINGS

(a) General requirements:

1. Radius of gyration about the axis $\perp$ to the plane of lacing $\nless$ radius of gyration about the axis in the plane of lacing.
2. The lacing system should not be varied throughout the length of the strut as far as practicable.
3. The single-laced systems on opposite sides of the main components should preferebly be in the same direction so that one be the shadow of the other.

(b) Design Specification:



(a) Single lacing (b) Double lacing

1. The angle of inclination of the lacing with the longitudinal axis of the column should be between 40° to 70°.
2. The slenderness ratio l_e/r of the lacing bars should not exceed 145. The effective length l_e of the lacing bars should be taken as follows:

Type of lacing	Effective length l_e
Single lacing, riveted at ends	Length between inner end rivet on lacingbar (= l, as shown in Fig. 17)
Double lacing, riveted at ends and at intersection	0.7 times length between inner end rivets on lacing bars (= 0.7 × l)
Welded lacing	0.7 times distance between inner ends of effective lengths of welds at ends (0.7 × l)

Lacing is generally preferred in case of eccentric loads.

Battening is normally used for axially loaded columns and where the components are not far apart.

3. For local Buckling Criteria

$\frac{L}{r^c_{min}}$ 50

0.7 whole section

where, L = distance between the centres of connections of the lattice bars to each component as shown in fig.

r^c_{min} = minimum radius of gyration of the components of compression member

4. Minimum width of lacing bars in rivetd construction should be as follows:

Nominal rivet diameter(mm)	22	20	18	16
Width of lacing bars(mm)	65	60	55	50

5. Minimum thickness of lacing bars:

$t \nless l/40$ for single lacing

$\nless l/60$ for double lacing riveted or welded at intersection

where, l = length between inner end rivets as shown in fig.

6. The lacing of compression members should be designed to resist a transverse shear, V = 2.5% of axial force in the member.

- For single lacing system on two parallel faces, the force (Compressive or tensile) in each bar,

$$F = \frac{V}{2\sin\theta}$$

- For single lacing system on two parallel faces, the force (Compressive or tensile) in each bar,

$$F = \frac{V}{4\sin\theta}$$

- If the flat lacing bars of width b and thickness t have rivets of diameter d then,

Compressive stress in each bar = $\frac{\text{force}}{\text{gross area}}$

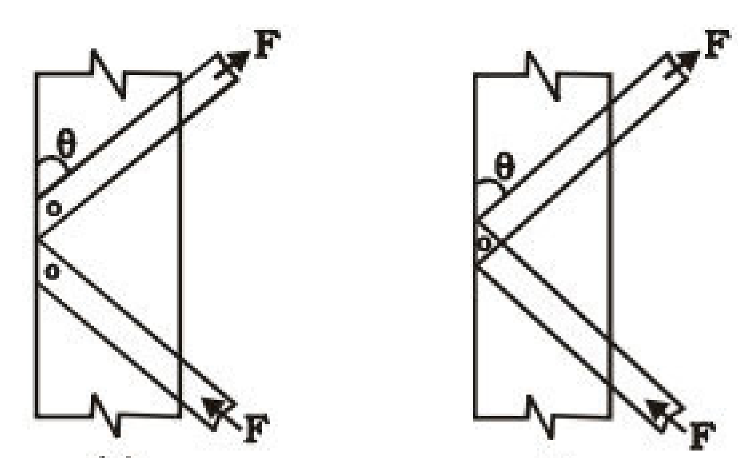
$$= \frac{F}{b \times t} \ngtr \sigma_{ac}$$

- Tensile stress in each bar = $\frac{\text{force}}{\text{net area}}$

$$= \frac{F}{(b-d) \times t} \ngtr \sigma_{at}$$

7. End Connections:

- ***Riveted connection:*** Riveted connections may be made in two ways as shown in Fig. (a) and (b).



(a) (b)

Force case (a),

Number of rivets required = $\frac{F}{\text{Rivet value}}$

For case (b),

- Number of rivets required = $\frac{2F\cos\theta}{\text{Rivet value}}$

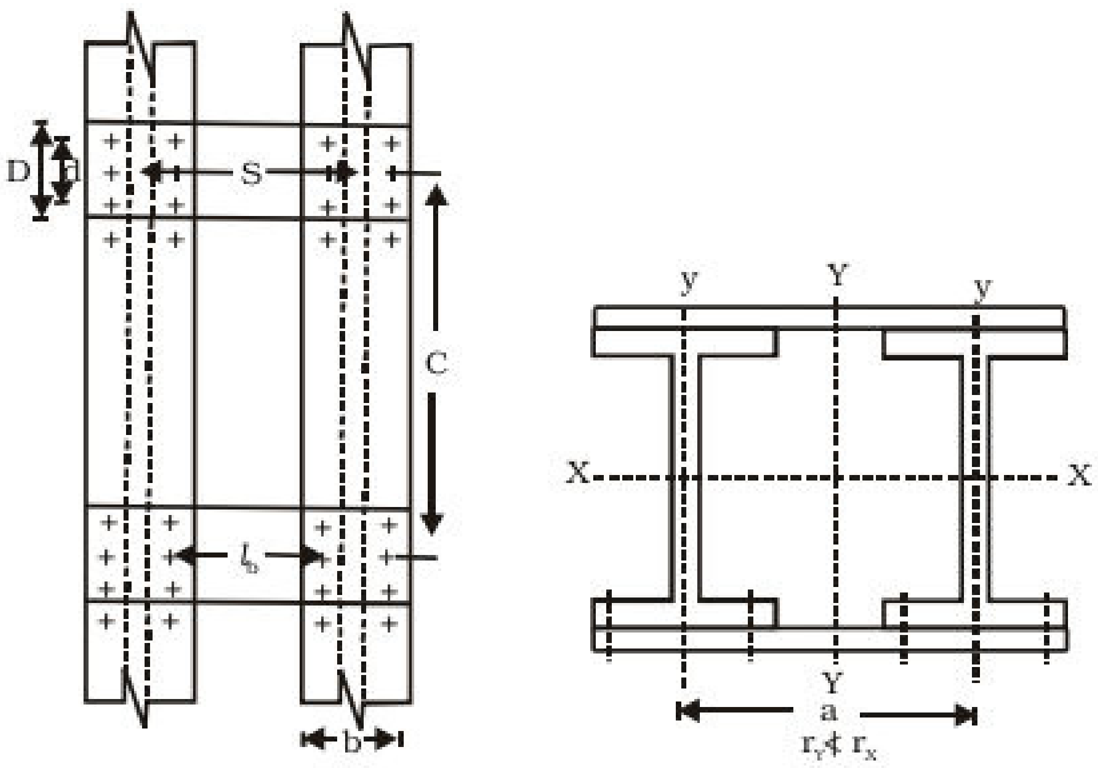
- **Welded connections**

Lap joint : Overlap $\nless$ (1/4) times thickness of bar or member, whichever is less.

Butt joint : Full penetration butt weld or fillet weld on each side. Lacing bar should be placed opposite to flange or stiffening member of main member.

BATTENS

(a) General Requirements:



1. $r_y \nless r_x$
2. The number of battens should be such that the member is divided into not less than three parts longitudinally.

(b) Design Specifications:

1. Spacing of battens C, from centre to centre of end fastening should be such that the slenderness ratio of the lesser main component,

$\frac{C}{r_{\min}^{c}} \ngtr$ 50, or 0.7 times the slenderness ratio of the compression member as a whole about $x-x$ axis (parallel) to battens, which is less
where, C = spacing of battens as shown in fig.

$r_{\min}^{c}$ = minimum radius of gyration of components.

2. $d > \left(\frac{3}{4}\right)a$ for intermediate battens,

d > a for end battens
and $d > 2 \times$ b for any batten.
where d = effective depth of batten,
a = centroid distance of members,
b = width of member in the plane of batten

3. Thickness of battens,

$t > \frac{l_b}{50}$ where, l_b = distance between innermost connecting line of rivets or welds.

4. $V = \frac{2.5}{100}P$ and P = total axial load on the compression member.

- Transverse shear V is divided equally between the parallel planes of battens. Battens and their connections to main components resist simultaneously a longitudinal shear.

$V_1 = \frac{V \times C}{N \times S}$ and a moment, $M = \frac{V \times C}{2N}$

where,
C = spacing of battens
N = number of parallel planes of battens
S = minimum transverse distance between centroids of rivet group or welding.

- Check for longitudinal shear stress,

$\frac{V_1}{D \times t} \ngtr \tau_{va}$

where, t_{va} = permissible avearge shear stress
= 100 MPa for steel of IS : 226-1975
D = overall depth of battens,
t = thickness of battens.

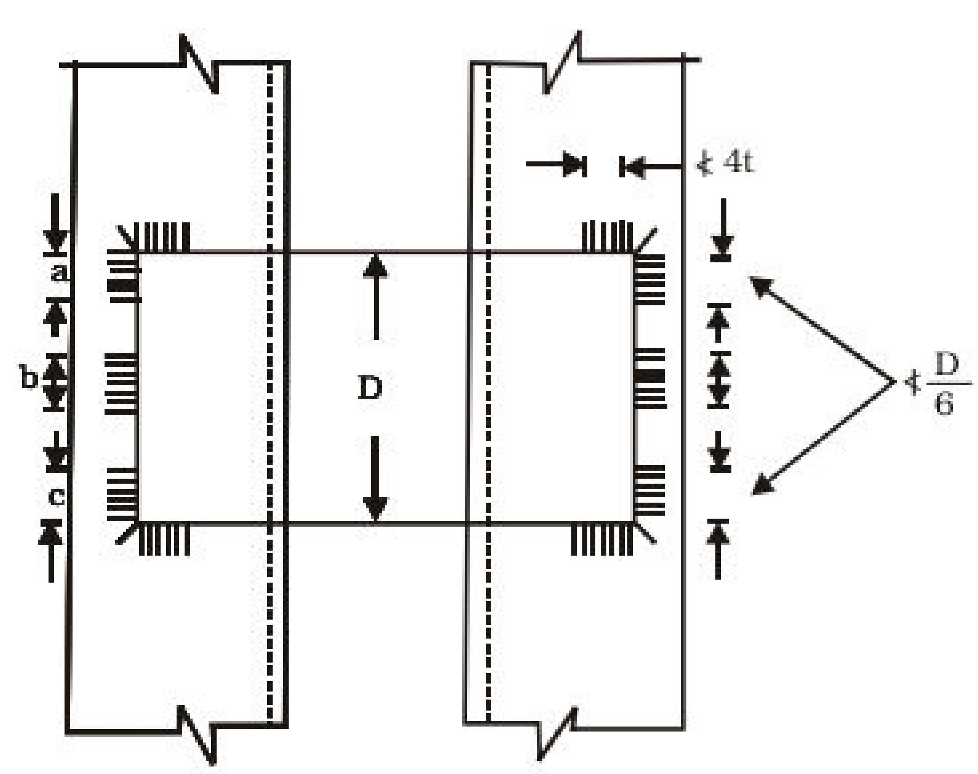
- Check for bending stress,

$\frac{M}{Z} = \frac{M}{\frac{1}{6}D \times t^2} \ngtr \sigma_{bc\,or\,bt}$

where, σ_{bc} σ_{bt} = permissible bending compressive of tensile) stress.
= 165 MPa for steel of IS : 226-1975.

5. End connections:

- Design the end connections to resist the longitudinal shear froce V_1 and the moment M as calculated in steep 4 above.
- For welded connections Lap 4t Where t is thickness of plate.
- Total length of weld at end of edge of batten D/2.
- Length of weld at each edge of batten 1/3 total length of weld required.
- Return weld along transverse axis of column 4t where, t and D are the thickness and overall depth of the battens respectively.



$a + b + c \nless \frac{D}{2}$ t = thickness of batten.,,

COLUMN BASES

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

Three types of column bares are usually used.

(i) Gusseted base

(ii) Grillage foundation

(iii) Slob base

GUSSETED BASE

A gusseted base consists of a base connected to the column through gusset plates. The thickness of the base plate in this case will be less than the thickness of the slob base for the same axial load as the bearing area of the column on base plate increased by the gusset plates.

As per IS: 800-1984 for column with gusseted base; gusset plates, angle cleats, stiffeners, fastenings, etc. in combination with the bearing area of the shaft should be sufficient to take the loads, bending moment and reaction to the base plate without exceeding the specified stresses. All bearing surfaces are machined to ensure perfect contact.

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

GRILLAGE FOUNDATION:

Grillage foundations are provided at shallow depths for columns carrying heavy loads on weak soils. It consists of two or more layers of steel beams completely encased in concrete as shown in e.g. below.

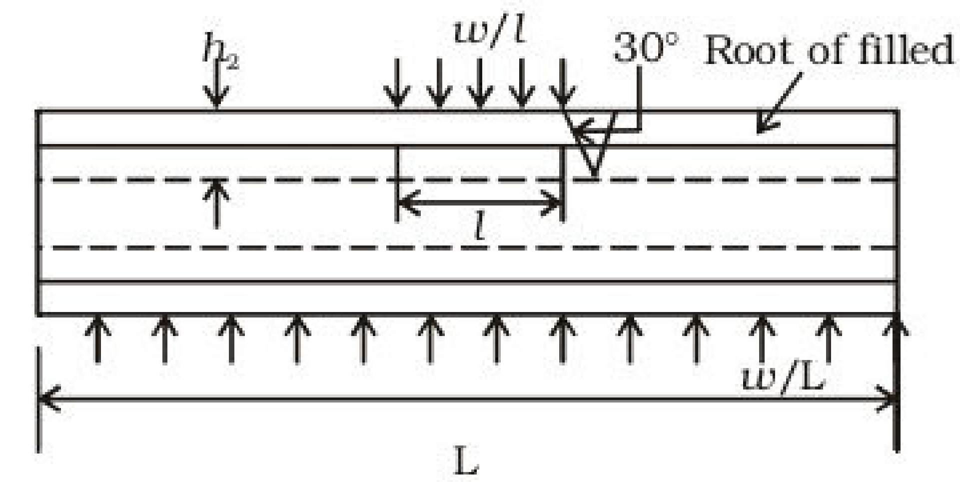
As per IS. 1800-1962 the permissible strees in grillage beams encased in concrete may be increased by 33.5% (50% when the effect of wind, seismic or erection load is also taken into account). If the following conditions are fullfilled.

(1) The beam are unpainted and are solidly encased in ordinary dense concrete with 10mm aggregate and of a works cube strength not less than 160 kgf/cm^2 at 28 days.

(2) Pipe seperators or their equivalent are used to keep the beams properly spaced apart so that the distance between the edges of adjacent flanges is not less than 75 mm.

(3) The thickness of the concrete cover on top of the upper flange at the ends, and at the outer edges of the sides of the outermost beams is not less than 100 mm.

Consider a tier of grillage beams having a length 'L' and carrying as shown in fig. The grillage beams of the tier are designed to withstand the mox. B.M. at the middle of the beams. They are further checked for maximum shear and web crippling.



The maximum Bending moment at the middle of a beam is

$$\frac{W(L-l)}{8}$$

The maximum shear force on the beam at the edge of the load is:

$$\frac{W(L-l)}{2L}$$

The bearing pressure on the web at the root of the filled should not exceeds the safe bearing pressure i.e., 1.33 × 189 MPa.

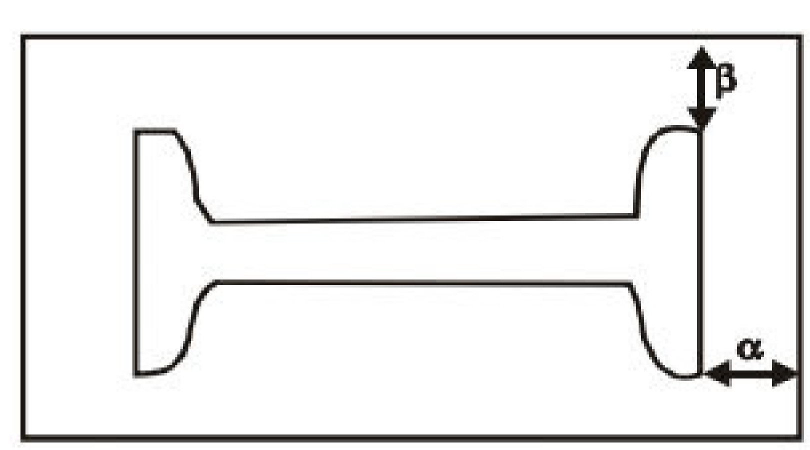
$$\therefore \frac{W}{\left(l+2\sqrt{3}h_2\right)t_w \times n} \le 1.33 \times 189\text{MP}a$$

where n = number of beams in tier

SLAB BASE

- Sufficient fastenings are provided reatin the column securely on the base plate and resist all moments and forces (except direct compression in the column) arising during transit, unloading and erection.
- Area of slab base = $\dfrac{\text{axial load in the column}}{\text{permissible compressive stress in concrete}}$
- The thickness of a rectangular slab base as per IS : 800-1984.

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



where,

t = the slab thickness (mm)

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

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

= max. (α, β).

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

= min. **(α, β)**

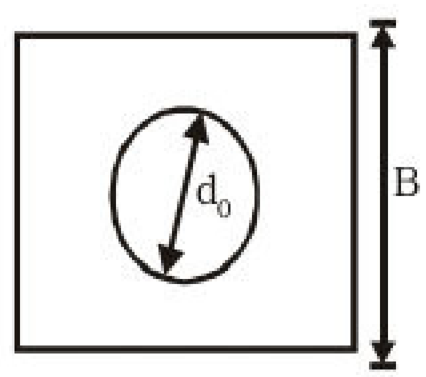
σ_{bs} = the permissible bending in slab bases

= 165 MPa for flanged beams

= 185 MPa for solid beams

- The thickness of a square slab base plate under a solid round column.

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



W = the total axial load (kN)

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

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

- The cap or base plate should not be less 1.5 (d_0 + 75) mm in length or diameter.

BEAMS

A beam is designed to resist maximum bending moment and is checked for shear stress and deflection, and also for web crippling and web bucking.

DESIGN FOR BENDING

- The maxiumum permissible compressive or tensile bending stress σ_{bc} or σ_{bt} = 0.66 f_y

  where, f_y = yield stress of steel

  The permissible bending stress (compressive or tensile) σ_{bc} or σ_{bt} as per IS: 226-1975 is follows:

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

EFFECTIVE LENGTH OF COMPRESSION FLANGE

(i) Effective Length of Compression Flange:

End Connections	**Effective length, *l***
(i) each endrestrained againt torsion.	
(a) ends of compression flange unrestrained for lateral bending	l = span
(b) ends of compression flange partially restrained for lateral bending	l = 0.85 × span
(c) ends of compression flange fully restrained for lateral bending	l = 0.7 × span
(ii) Cantilever bneams of projecting length L,	
(a) Built-in at the support, free at end	l = 0.85L
(b) Built-in at the support, restrained against torsion at the end by continuous construction.	l = 0.75L
(c) Bult-in at the support, restrained against lateral deflection and torsion at the free end by continuous cross members over several beams	l = 0.5L
(d) Continuous and unrestrained against torsion at the support and free at the end	l = 3L
(e) Continuous and partially restrained against torsion at the support and free at end	l = 2L
(f) Continuous at the support, restrained against torsion at the support ant free at the end	l = L

The above values are incresed by 20% if the ends of beam are not restrained against torsion.

If there is a degress of fixity at the end, the dffective length should be multiplied by $\frac{0.5}{0.85}$ in (b) and (c) above and by $\frac{0.75}{0.85}$ in (d),(e) and (f) above.

(ii) Check for Shear

- Max permissible, shear stress

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

- For design purpose, the above condition is deemed to be satisfied if the average stress in an unstiffened member calculated on the cross section of web does not exceed the value.

$$\tau_{va} = 0.4 f_y$$

(iii) Check for Deflection

- The maximum deflection $\not> \frac{l}{325}$ of the span in general.

(iv) Check for web crippling and web buckling:

BUILT UP BEAMS

(i) Symmetrical built-up beams

- Area of each cover plate

$A_p = \frac{Z - Z_1}{d}$ where, Z_1 = Section modulus of rolled I section available, d = depth of beam

(ii) Unsymmetrical built-up beam

- The area of cover plates $A_p = \frac{1.2 \times (Z - Z_1)}{d}$

GANTRY GIRDERS

(i) The gentry girders are subjected to unsymmetrical bending due to lateral thrust.

(ii) The deflection of gantry girders under dead and imposed loads should not exceed the following values as per IS : 800- 1984

(a) Where cranes are manually operated $\frac{L}{500}$

(b) Where electric overhead travelling cranes are operated, over 50t $\frac{L}{750}$

(c) Where electric overhead travelling cranes are operated, over 50t $\frac{L}{1000}$

(d) Other moving loads such as charging cars etc. $\frac{L}{600}$

Where, L = span of crane runway girder.

BEAM COLUMN

- Members subjected to axial compression and bending are proportional to satisfy the Eq. (1)

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx} \times \sigma_{bcx,cal}}{\left[1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right]\sigma_{bcx}} + \frac{C_{my} \times \sigma_{bcy,cal}}{\left[1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccy}}\right]} \leq 1.0 \quad ..(i)$$

However if the ratio $\frac{\sigma_{ac,cal}}{\sigma_{ac}}$ is less than 0.15 Eq (ii) may be used in lieu of Eq. (i)

$$\frac{ac,cal}{ac} \quad \frac{bcx,cal}{bcx} \quad \frac{acy,cal}{bcy} \quad 1.0$$

PLATE GIRDERS

- Economic depth of the girder

$$D = 1.1\sqrt{\frac{M}{_{bt} \quad t_w}}$$

- A self weight may be assumed to begin with the design

$$W = \frac{W}{300}\ \text{kN/m}$$

DESIGN OF WEB

- Average shear stress in the web $_{va,cal} \quad \frac{V}{d_w \quad t_w}$

permissible average shear stress, $_{va}$.

WEB STIFFENERS

- IS: 800-1984 recommends the provision of web stiffeners as follows:

(i) $\frac{d_1}{t_w} \leq$ lesser of $\frac{816}{\sqrt{\tau_{va,cal}}}$ and $\frac{1344}{\sqrt{f_y}}$ and 85. No stiffener is required.

(ii) $\frac{d_2}{t_w} \leq$ lesser of $\frac{3200}{\sqrt{f_y}}$ and 200, Vertical stiffeners are provided.

(iii) $\frac{d_2}{t_w} \leq$ lesser of $\frac{4000}{\sqrt{f_y}}$ and 250.

Vertical stiffeners and one horizontal stiffener at a distance from the compression flange equal to two-fifths of the distance from the compression flange to the neutral axis are provided.

(iv) $\frac{d_2}{t_w} \leq$ lesser of $\frac{6400}{\sqrt{f_y}}$ or 400.

where, $d_2 = 2 \times$ clear distance from compression flange angles or plate or tongue plate to the neutral axis.

- In no case should the greater clear dimension of a web panel should exceed $270t_w$ nor the lesser clear dimension of the same panal should exceed $180\ t_w$.
- The term $\left(A_f + \frac{A_w}{6}\right)$ is called the effective flange area.

PERMISSIBLE BENDING STRESS

- The maximum compressive stress $\sigma_{bc'cal}$ is calculated on gross flange area, i.e.,

$\sigma_{bc'cal} = \dfrac{M \times D/2}{I_{gross}} \not> $ permissible bending stress in compression, σ_{bc}

- The maximum tensile stress $\sigma_{bt,cal}$ is calculated on the net flange area i.e.,

$\sigma_{bc'cal} = \dfrac{M \times D/2}{I_{gross}} \times \dfrac{\text{gross flange area}}{\text{net flange area}} \not>$ permissible bending stress in tension, σ_{bt}.

- The maximum tensile stress $\sigma_{bt'cal}$ is calculated on the net flange area i.e.,

$\sigma_{bt,cal}$.

$= \dfrac{M \times D/2}{I_{gross}} \times \dfrac{\textit{gross flange area}}{\textit{net flange area}} \not>$ permissible bending stress in tension, σ_{bt}.

CURTAILMENT OF FLANGE PLATES

- Length of the plate to be curtailed

$$l_n = l\sqrt{\frac{A_1 + A_2 + A_3 + \ldots + A_n}{A_f + A_{we}}}$$

Where,

l = span

n = no of plates to be curtailed counting 1, 2, 3,.... from outer plate.

A_{we} = effective web area

WEB STIFFENERS

- Unless the outer edge of each stiffener is continuously stiffened, the outstand of all stiffners from the web should not be more than $\dfrac{256 \quad t}{\sqrt{f_y}}$

(= 16t for steel sections and 12t for flats where t is the thickness of the section or flat.

- Where vertical stiffeners are required, they should be provided throughout the length of the girder at a distance not greater than $1.5d_1$ and not less than $0.33\ d_1$.
- When horizontal stiffeners area provided d_1 should be taken as the clear distance between the horizontal stiffener and tension flange (farthest flange) ignoring fillets.
- The moment of inertia I of a pair of vertical stiffener.

about the centre of web or a single stiffener about the face of the web should be, $I \geq \dfrac{1.5 \times d_1^3 \times t^3}{C^2}$

where,

t = the min. required thickness of web.

c = the max. permitted clear distance between vertical stiffener for thickness t.

- Sometimes vertical stiffeners are subjected to external forces and therefore the moment of inertia of the stiffener should be increased as described below.

(a) Bending moment on stiffener due to eccentricity of vertical loading with respect to vertical axis of the web.

Increase of $I = \dfrac{150M \times D^2}{E \times t_w} cm^4$

(b) Lateral loading on stiffener :

Increase of $I = \dfrac{0.3V \times D^3}{E \times t_w} cm^4$

- For first horizontal stiffener at 2/5th of the distance between compression flange and neutral axis, from the compression flange :

$$I \geq 4C \times t^3$$

where,

I = moment of inertia of a pair of horizontal siffeners about the centre of the web or single stiffener about the face of the web

t = the minimum thickness of web required

c = actual distance between vertical stiffeners

- For second horizontal stiffener at the neutral axis,

$$I \geq d_2 \times t^3$$

Stiffeners are connected to web to withstand a shearing force not less than $\dfrac{125 \times t_w^2}{h}$ kN/m,

where, h = outstand of stiffener in mm.

LOAD BEARING STIFFENERS

- Bearing stiffeners are provided at the points of concentrated loads and at supports.
- Where these stiffeners are to provide restraint against torsion of the plate girder at the ends,

$$I \quad \frac{D^3 \quad T}{250} \quad \frac{R}{W}$$

INDUSTRIAL ROOFS

If loads from purlins, false ceiling etc. are applied in between the nodes, then principal rafters or main ties are designed for combined stresses from bending and axial load.

PURLINS AND GRITS

- Angle, channel, I and Z sections are used for purlins and grits to support the cladding.

- IS : 800 - 1984 provides general design procedure for angle purlins conforming to steel grades Fe 410-O, Fe 410-S, Fe 410 -W and roof slopes not exceeding 30° based on a minimum live load of 750 N/m^2 if the following requirements are fulfilled :

  (i) Width of angle leg in the plane perpendicular to the roof covering $\geq \frac{L}{45}$.

  (ii) Width of angle leg in the plane parallel to the roof covering $\geq \frac{L}{60}$.

  (iii) Maximum bending moment in the purlin,

$$M = \frac{w \times L^2}{10}$$

  Where,
  w = uniformly distributed load per unit length on purlin including wind load
  L = span of purlin.
  (iv) The bending moment about minor axis may be neglected and the angle purlin may be designed for the above moment.

$$\therefore \qquad Z_x \text{required} = \frac{M}{\sigma_{bc}} = \frac{w \times L^2}{10 \times 165} mm^3$$

ROFF TRUSS:-

Figure below should some of the roof trusses used for industrial sheds. They are economical for spans more than 6m. Pratt, However and compound Fink trusses are used upto maximum span of 30m. The pitch of a roof truss (= rise/span) should be $\frac{1}{4}$ to $\frac{1}{6}$ for proper drainage. Spacing of roof trusses is kept $\frac{1}{3}$ to $\frac{1}{5}$ of the span. Roof trusses usually require very light members. However a minimum angle section ISA 50×50×6 mm should be provided to avoid damage during transportation, erection etc. Double angle sections are usually used for main rafter and ties. The gusset plate should be at least 6 mm thick and at least 2 rivets should be used to connect any member to it.

Members of roof trusses are axially loaded tension or compression member if they are slender and their resistance to bending is neglected.

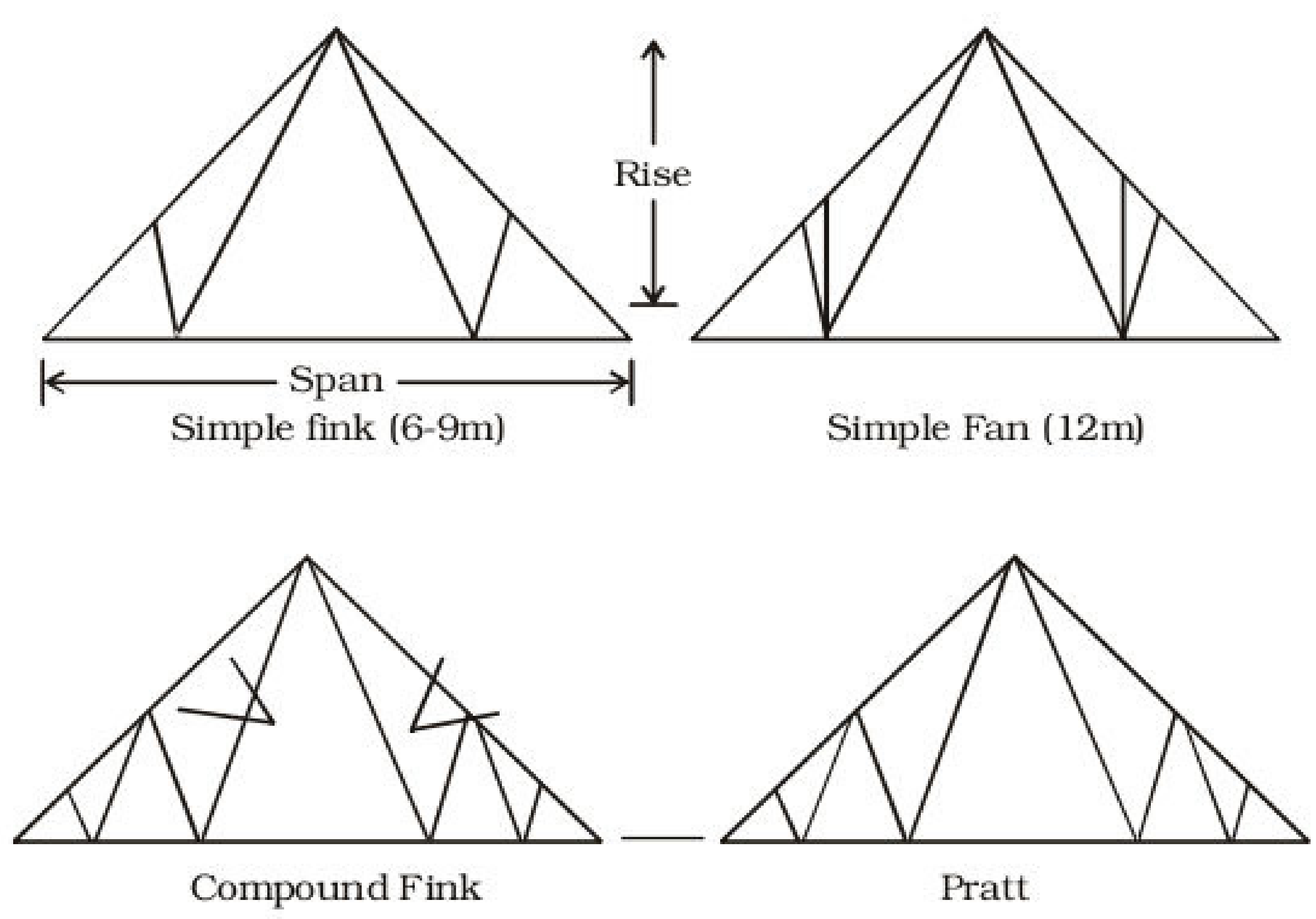


Fig: Various Type of Roof Trusses

However, if loads from purlins, false ceiling etc. are applied in between the modes, then principal rafters or main ties are designed for combined stresses from bending and axial load. ❑❑❑