Design of Steel Structures

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Rivets, Bolts and Welds

Structural Fasteners

Riveting

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

- Gross dia of rivet or dia of hole
  d' = d + 1.5 mm for d ≤ 25 mm
  and d' = d + 2.0 mm for d ≥ 25 mm
  where d = Nominal dia of rivet
  d' = Gross dia of rivet or dia of hole…

- Unwins formula
  $d_{mm} = 6.05\sqrt{t_{mm}}$
  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:

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 x d x *l* where d = diameter, and *l* = length of the bolts.

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.

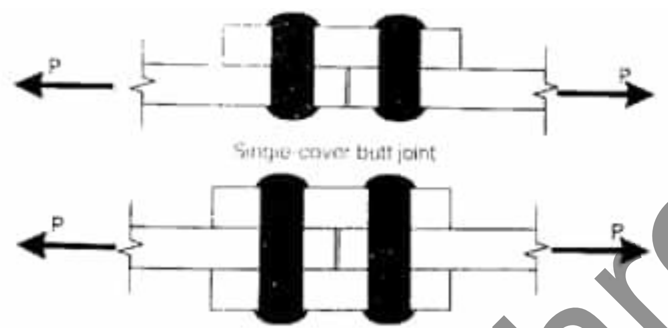
(i) Lap joint

- 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.



Double-cover butt joint

- The butt joint may have a single row or staggered or chain riveting.

Failure of Riveted/Bolted Joints

i. **By Tearing of Plate between rivets**
Strength of tearing per pitch length
$P_t = (p - d')\, t \times f_t$
where, f_f = 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**

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

iii. **Strength of rivet in double shear**

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

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

iv. **Failure due to bearing of crushing of rivet of plates**
Strength of rivet in bearing
$P_b = f_b . d'^2 . t$ where, f_b = bearing strength of rivet.

Efficiency of Joints

(η)

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

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 for rivet holes

= p. t. f_t

- Rivet value

$$R_v = \text{minimum} \begin{cases} P_s \\ P_b \end{cases}$$

- Number of rivet,

$$n = \frac{\text{Force}}{R_v}$$

IS 800: 1984 Recommendation

Maximum permissible stress in rivets & bolts

Types of fastener	Axial tension, $\sigma_{at}(MPa)$	Shear, τ_{vf} (MPa)	Bearing, $\sigma_{pf}(MPa)$
Power driven			
(a) Shop rivets	100	100	300
(b) Filed 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)	32 t or 300 mm, Whichever is less
(ii) rivets lying in a line parallel to the force in the member:	
(a) in tension	16 t or 200 mm. whichever is less
(b) in compression	12 t or 200 mm. whichever is less

Where t = thickness of thinner outside plate

Permissible Stresses

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

Max Permissible Deflections

a. Max permissible horizontal and vertical deflection

$$= \frac{\text{Span}}{325}(LSM)$$

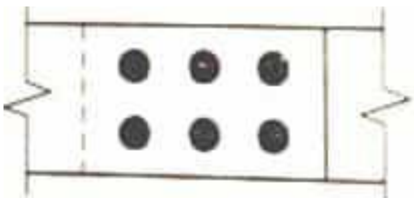
b. Max permissible deflection when supported elements are susceptible to cracking

a. Max permissible deflection when supported elements are not susceptible to cracking

$$= \frac{\text{Span}}{360}(LSM)$$

Arrangement of Rivets

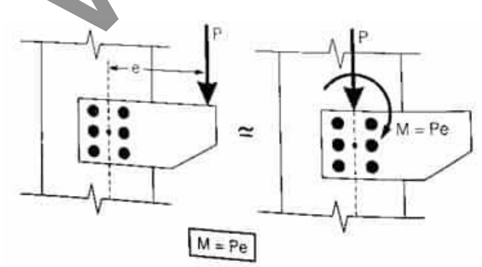
(a) Chain Riveting



(b) Diamond Riveting

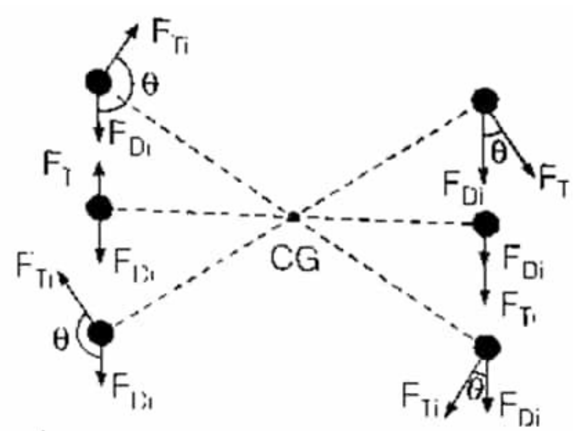
(c) Staggered Riveting

Eccentric Connections



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

$$(ii)\ F_{Ti} = \frac{Per_i}{\sum A_i r_i^2} A_i$$



$$(iii)\ Fr_i = \sqrt{(F_{Di})^2 + (F_{Ti})^2 + 2F_D.F_T \cos\theta} \leq R_v$$

Where, F_{Di} = Direct force in i^{th} rivet.

F_{Ti} = Force in i^{th} rivet due to torsional moment

r_i = Distance of i^{th} rivet from CG

A_i = Area of i^{th} rivet

$$= \frac{\pi}{4}(d_i)^2$$

F_{Di} = Always acts in the direction of applied load P.

F_{Ti} = Always acts perpendicular to the line joining CG of rivet group and the rivet under consideration.

F_{ri} = Resultant force in i^{th} rivet.

Angle b/w 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

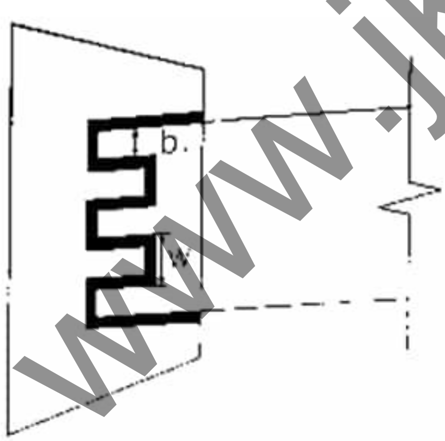
Minimum size of weld

It depends upon thickness of thicker plate

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

Max clear spacing between effective length of weld in compression zone = 12t or 200 mm (minimum). In tension zone = 16 t or 200 mm (minimum)

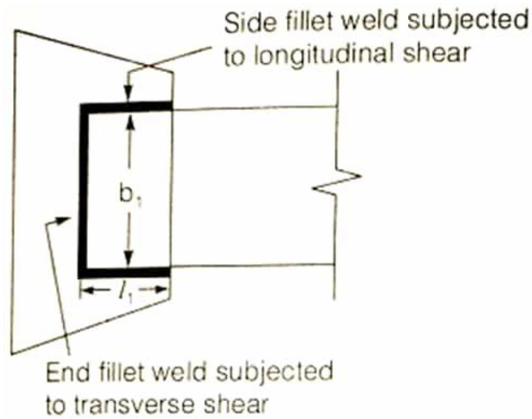
- **Slot weld**



$b_1 \not< 2t$

$w \not< 3t \text{ or } 25mm$

- **Slide fillet weld**



(a) $I_1 \not< b_1$

(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 $N/mm^2 \cong 100$ N/mm^2
Throat thickness t = k x size of weld

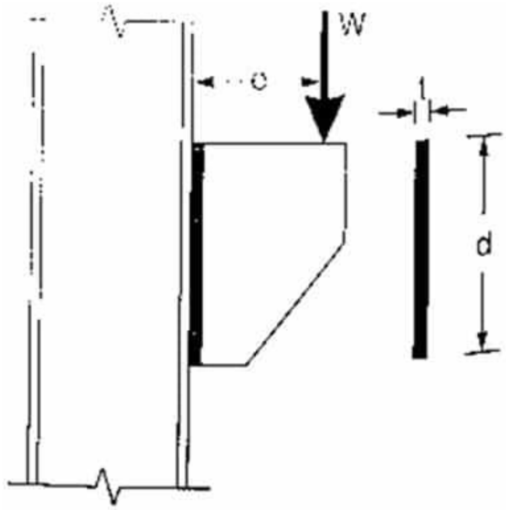
- **Butt-welded Joint Loaded Eccentrically**

Let the thickness of weld throat = t, and length of weld = d

- Shear stress at weld,

$$P_s = \frac{W}{d \times t}$$

Where t = thickness of weld throat and d = length of weld.



- Tensile or compressive stress due to bending at extreme fibre,

$$P_b = \frac{6M}{t \times d^2}$$

For the safety of joint the interaction 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 = 0.67f_y

For solid section

(□,○,△)

permissible bending stress is 185 N/mm^2

Fillet-Welded Joint Loaded Eccentrically

There can be two cases:

- Load not lying in the plane of the weld
- Load lying in the plane of the weld

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

- Let thickness of weld throat = t and total length of weld = 2 x d
- Vertical shear stress at weld,

$$p_s = \frac{W}{2d \times t}$$

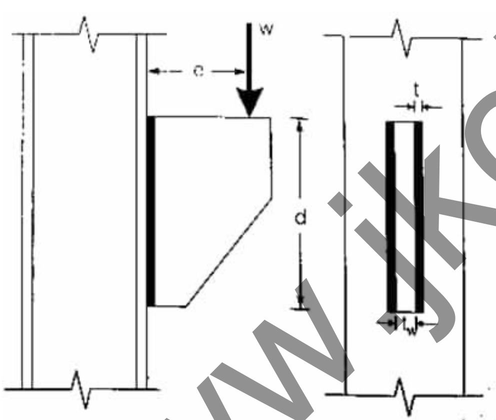
- Horizontal shear stress due to bending at extreme fibre,

$$p_b = \frac{M}{I} \times y = \frac{(W \times e) \times d / 2}{\frac{2 \times t \times d^3}{12}} = \frac{3We}{td^2}$$

- Resultant stress,

$$p_r = \sqrt{p_s^2 + p_b^2}$$

- 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

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

(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,

$$p_s = \frac{W}{l \times t}$$

where,

$l(l_1 + l_2 + l_3) =$

the length of weld and t = thickness of the throat

- Torsional stress due to moment, at any point in the weld,

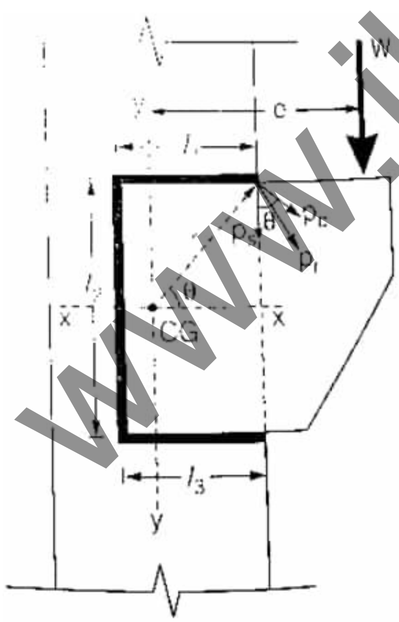
$$p_b = \frac{T \times r}{I_p}$$

where,

T = torsional moment = W x e

r = distance of the point from cg of weld section

I_p = polar moment of inertia of the weld group = $I_x + I_y$



- The resultant stress,

$$p_r = \sqrt{p_s^2 + p_b^2 + 2p_s p_b \cos\theta}$$

- For safety,

$$p_r \ngtr$$

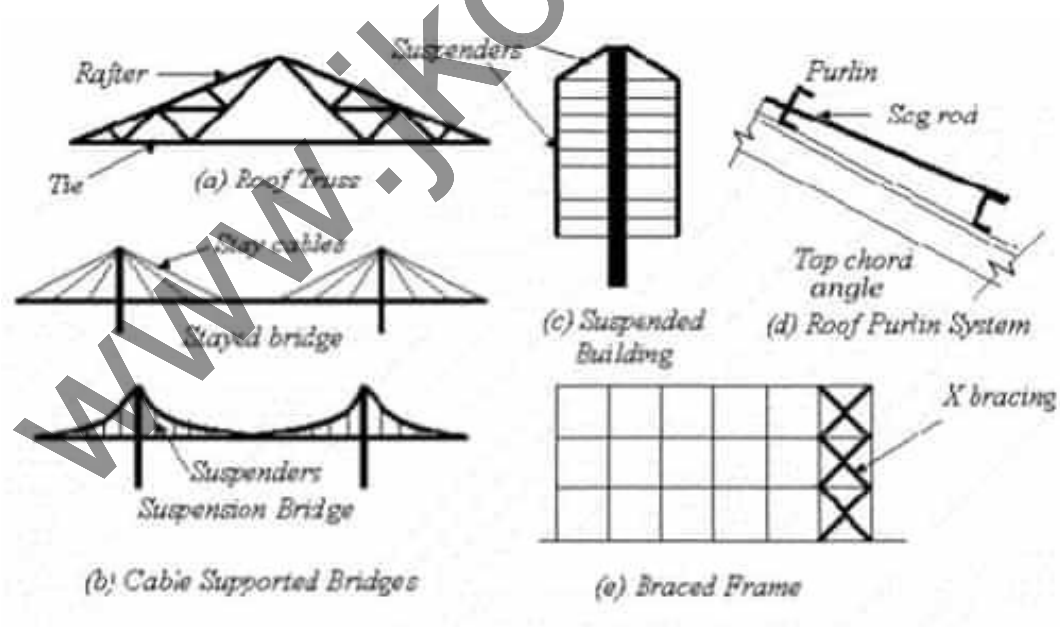
permissible stress in fillet weld, i.e. 108 MPa.

- The resultant stress p_r will be maximum at a point where r is maximum and q is minimum.

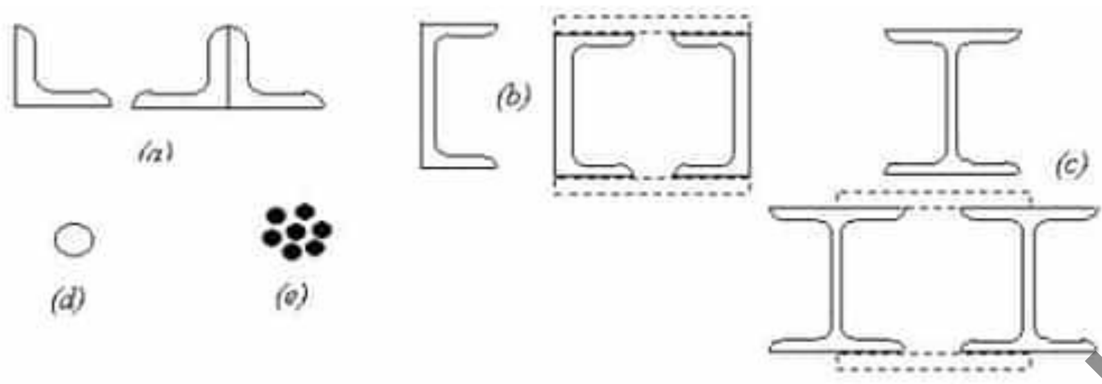
Tension Members

Tension Member

Tension members are linear members in which axial forces act so as to elongate (stretch) the member. A rope, for example, is a tension member. Tension members carry loads most efficiently since the entire cross section is subjected to uniform stress. Unlike compression members, they do not fail by buckling. Ties of trusses, suspenders of cable stayed and suspension bridges, suspenders of buildings systems hung from a central core (such buildings are used in earthquake prone zones as a way of minimizing inertia forces on the structure), and sag rods of roof purlins are other examples of tension members.



Tesnsion Members in Structures



Cross Sections of Tension Members

Introduction

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

Types of member	Max. Slenderness Ratio
1. A tension member in which reversal of direct stress due to loads other than 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

Net area = (b x 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.

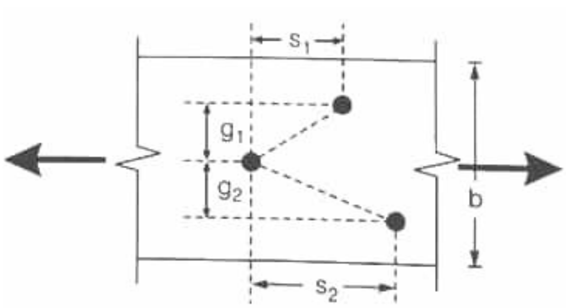
g_1 = Distance between two consecutive rivets perpendicular to the direction of load also called gauge.

b = Width of the plate

n = Number rivets at the section

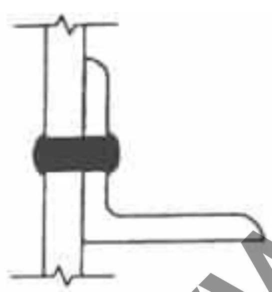
T = Thickness of the plate

d' = Gross diameter of the rivet



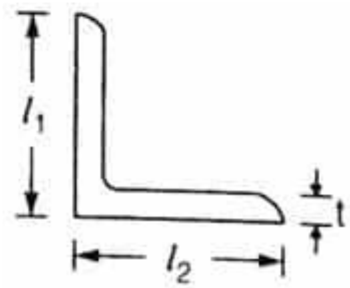
(ii) Single angle connected by one leg only.

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



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

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



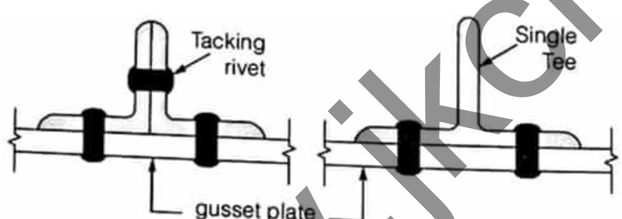
(b)

(c) $A_1 = \left(t_1 - \frac{t}{2}\right)t$

(d) $A_2 = \left(t_2 - \frac{t}{2}\right)t$

(e) $A_{net} = (I_1 + I_2 - t)t$

(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.



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

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

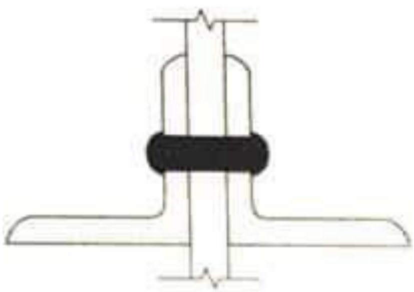
where, A_1 = Area of connected leg

A_2 = Area of outstand (unconnected leg)

(c) The area of a web of tee = Thickness of web x (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 places 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 $k = \frac{3A_1}{3A_1 + A_2}$

Permissible Stress in Design

- The direct stress in axial tension on the effective net area should not exceed σ_{at}
  where $\sigma_{at} = 0.6f_y$
  and f_y = minimum yield stress of steel in MPa

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 Members

Strength of an Axially Loaded Compression Member

- The maximum axial compressive load P

P = σ_{ac} x A where, P = axial compressive load (n)

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

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

IS800-1984 uses the Merchant Rankine formula for σ_{ac} which is given as

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

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

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

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 x 10^5 MPa

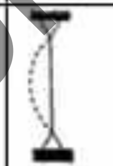
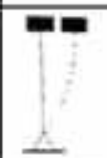
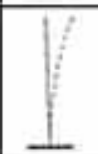
n = a factor assumed as 1.4

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

S.No.	Type of Member	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 constant dimensions (Clause 5.2.2 IS: 800-1984)

S.No.	Degree of end restraint of compression member	Recommended value of effective Length	Symbol
1.	Effectively held in position and restrained against rotation at both ends	0.65 L	
2.	Effectively held in position at both ends restrained against rotation at one end	0.80 L	
3.	Effectively held in position at both ends, but not restrained against rotation	1.00 L	
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.	1.20 L	
5.	Effectively held in position and restrained against rotation at one end, and at the other end partially restrained against rotation	1.50 L	
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	2.00 L	
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	2.00 L	

Angle struts

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

Table: Angle Struts (Clauses 5.5, IS: 800 – 1984)

S.No.	Type	End Connections	Effective length	Allowable stress	Slenderness Ratio
1.	Single angle discontinuous	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	Connected on same side of gusset plate One rivet or bolt at each end Two or more rivets, bolts or welding at each end (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}	$\frac{l}{r} \not> 180$
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}	

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.

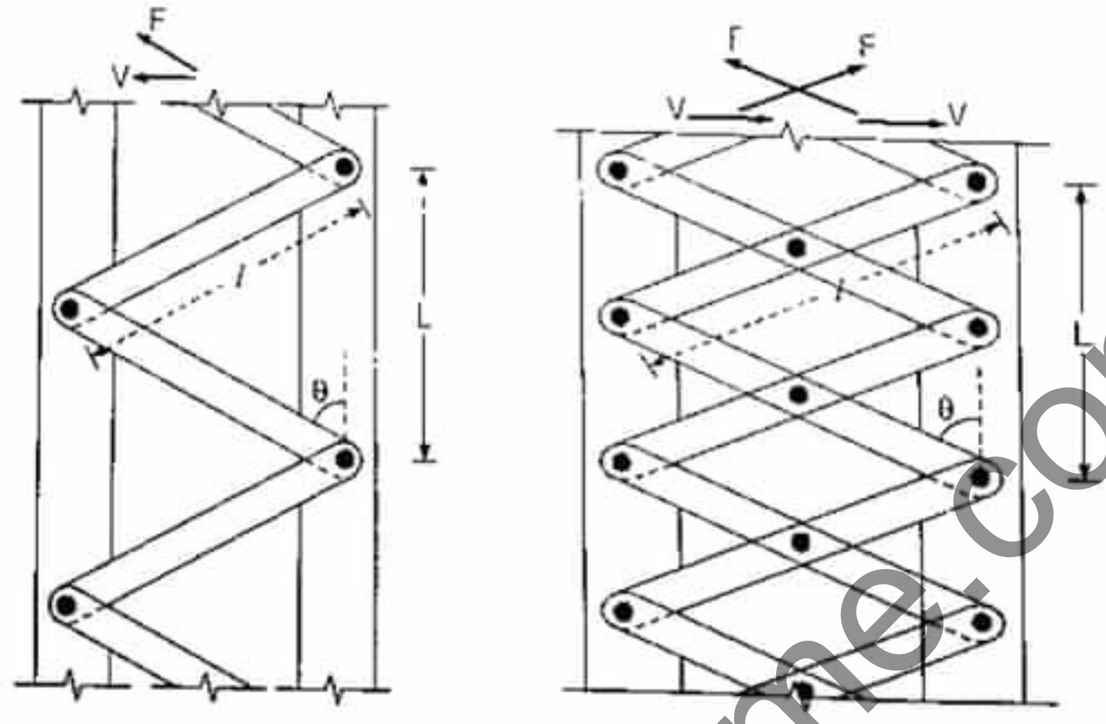
(v) Check the slenderness ratio.

Lacings

(a) General requirements:

1. Radius of gyration about the axis ⊥ to the plane of lacing $\not<$ radius of gyration about the axis in 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 system on opposite sides of the main components should preferably be in the same direction so that one be the shadow of the other.

(b) Design Specification:



(a) Single lacing (b) Double lacing

- The angle of inclination of the lacing with the longitudinal axis of the column should be between 40° to 70°.
- 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 I_e**
Single lacing, riveted at ends	Length between inner and rivets on lacing bar (= I, as shown in Fig.)
Double lacing, riveted at ends and at intersection	0.7 times length between inner end rivets on lacing bars (= 0.7 × I)
Welded lacing	0.7 times distance between inner ends of effective lengths of welds at ends (0.7 × I)

- For local Buckling criteria

$$\frac{L}{r^c_{min}} \not> 50$$

$$\not> 0.7\lambda_{wholesection}$$

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

- Minimum width of lacing bars in riveted construction should be as follows:

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

- Minimum thickness of lacing bars:

$t \nless I / 40$

for single lacing

$\nless I / 60$

for double lacing riveted or welded at intersection

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

- The lacing of compression members should be designed to resist a transverse shear, V = 25% of axial force in the member.
  - For a single lacing system on two parallel faces, the force (compressive or tensile) in each bar,

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

  - For double lacing system on two parallel planes, 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,

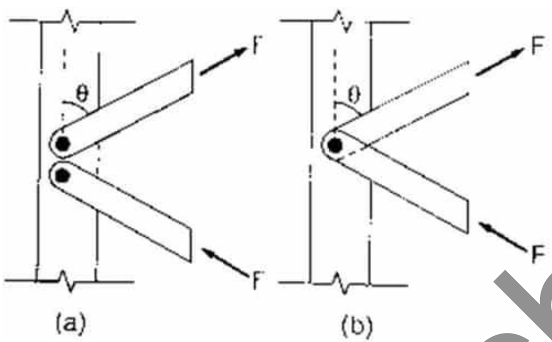
- Compressive stress in each bar

$$= \frac{\text{force}}{\text{gross area}} = \frac{F}{b \times t} \ngtr \sigma_{ac}$$

- Tensile stress in each bar

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

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



For case (a),

Number of rivets required

$$= \frac{\text{F}}{\text{Rivet value}}$$

For case (b),

- Numbers of rivets required

$$= \frac{\text{2Fcos}}{\text{Rivet value}}$$

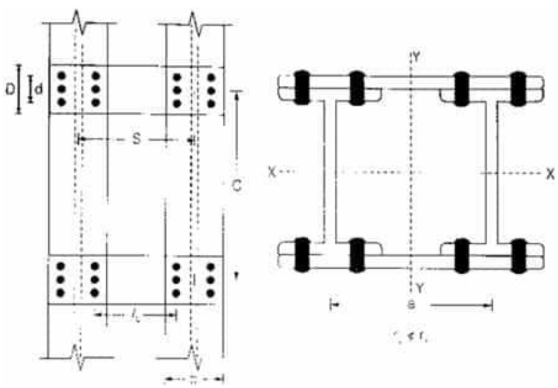
- **Welded connections**
  Lap joint: Overlap $\nless (14)$

times thickness of bar or member, whichever is less.

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

Battens

(a) General Requirements:



- $r_y \not< r_x$
- 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,

   $$\frac{C}{r^c_{min}} \not> 50, or 0.7$$

   times the slenderness ratio of the compression member as a whole about x – y axis (parallel to battens), which is less
   where C = spacing of battens as shown in fig.
   $r^c_{\min} =$

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 × 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 of welds.

4.

$$V = \frac{2.5}{100}P$$

and P = total axial load on the comp. 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.

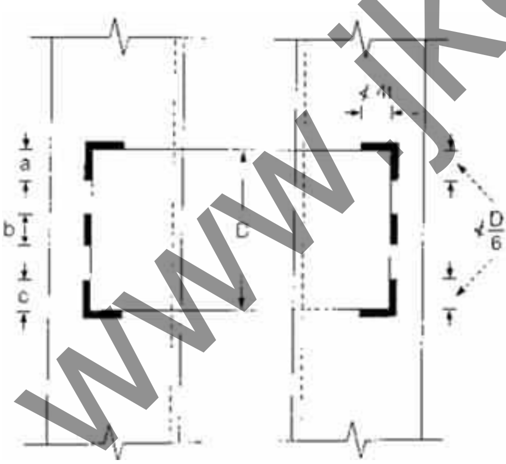
- Check for longitudinal shear stress,

  $\frac{V_1}{D\times t} \not> \tau_{va}$ where, t_{va} = permissible average shear stress
  = 100 MPa for steel of IS : 226-1975
  D = overall depth of battens,
  t = thickness of battens.
- Check for bending stress,

  $$\frac{M}{Z} = \frac{V_1}{\frac{1}{6}D\times t^2} \not> \sigma_{bc\,or\,bt}$$

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

5. End connections:
   - Design the end connections to resist the longitudinal shear force V_1 and the moment M as calculated in steep 4 above.
   - For welded connections Lap $\not< 4t$ where t is thickness of plate
   - Total length of weld at end of edge of batten $\not< D/2$
   - Length of weld at each edge of batten $\not< 1/3$ total length of weld required
   - Return weld along transverse axis of column $\not> 4t$ where, t and D are the thickness and overall depth of the battens respectively.



$$a+b+c \not< \frac{D}{2}$$

t = thickness of batten

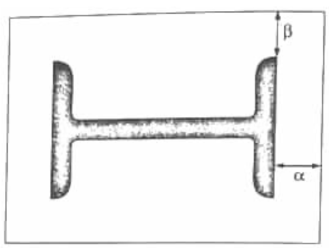
Slab Base

- Sufficient fastenings are provided to retain 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

$$= \frac{\text{axial load in the column}}{\text{permissible compressive stress in concrete}}$$

- 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. (∝, β).

b = the lesser projection of the plate beyond the column (mm) = max. (∝, β).

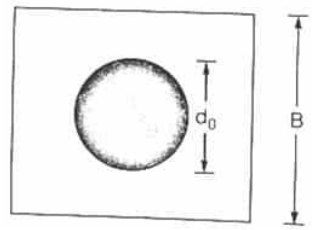
σ_{bs} = the permissible bending stress in slab bases

= 165 MPa for flanged beams

= 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.

Beams & Plate Girder

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 buckling.

Design for Bending

Nominal plate thickness	Yield stress f_y (MPa)	$S_{bc} = S_{bt}$ (MPa)
Angle, tee, I, channel and flat section Up to and including 20 mm	250	165
Over 20 mm up to and including 40 mm	240	158.4

• **Cantilever beams 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) Built-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 and free at the end	l = L

Effective Length of Compression Flange

End Connections	**Effective length, *l***
(i) each end restrained against 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 x span
(c) ends of compression flange fully restrained for lateral bending	l = 0.7 x span

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 shear stress in an unstiffened member calculated on the cross section of web does not exceed the value
  $\tau_{va} = 0.4\ f_y$

Built-up Beams

- **Symmetrical built-up beams**
  - each cover plate

$$A_p = \frac{Z - Z_1}{d}$$

where, Z_1 = Section modulus of rolled I section available.
d = depth of beam

- **Unsymmetrical built-up beam**
  - The area of cover plates

$$A_p = \frac{1.2 \times (Z - Z_1)}{d}$$

Gantry Girders

(a) Where cranes are manually operated	$\frac{L}{500}$
(b) Where electric overhead travelling crane are operated upto 50*	$\frac{L}{750}$
(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

Axial Compression + Bending

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

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

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)

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma_{bcx,cal}}{\sigma_{bcx}} + \frac{\sigma_{bcy,cal}}{\sigma_{bcy}} \leq 1.0 \qquad \text{........(ii)}$$

Plate Girders

- Economic depth of the girder $D = 1.1\sqrt{\frac{M}{\sigma_{bt} \times t_w}}$

Design of Web

- Average shear stress in the web $\tau_{va,cal} = \frac{V}{d_w \times t_w} \not> $ permissible average shear stress, τ_{va}

Web stiffeners

- $\frac{d_1}{t_w} \leq$ lesser of $\frac{816}{\sqrt{\tau va,cal}}$ and $\frac{1344}{\sqrt{f_y}}$ and 85. No stiffener is required.
- $\frac{d_2}{t_w} \leq$ lesser of $\frac{3200}{\sqrt{f_y}}$ $and\, 200.$ Vertical stiffeners are provided.
- $\frac{d_2}{t_w} \leq$ lesser of $\frac{4000}{\sqrt{f_y}}$ $and\, 250.$ Vertical
- $\frac{d_2}{t_w} \leq$ lesser of $\frac{6400}{\sqrt{f_y}}$ $or\, 400.$

Permissible Bending Stress

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

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

Curtailment of Flange Plates

- Length of the plate to be curtailed

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

Where, l = span n = no. of plates to be curtailed counting 1, 2, 3,... from outer plate.
A_{we} = effective web area

Web Stiffeners

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

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

- Lateral loading on stiffener:

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

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 $\frac{125 \times t_w^2}{h}$ 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,

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