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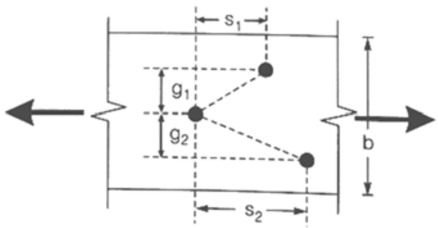
Short Notes on Design of Steel Structures

Tension Member

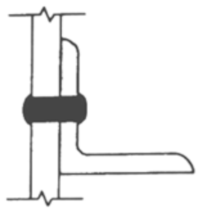
- A tension member in which reversal of direct stress due to loads other then wind or earthquake forces has maximum slenderness ratio =180
- A member normally acting as a tie in roof truss or bracing system. But subjected to possible reversal of stress resulting from the action of wind or earthquake forces has maximum slenderness ratio =350

Net Sectional Area

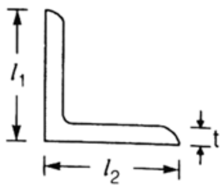
- For plate: Net area = (b x t) – nd't $+\left(\frac{s_1^2}{4g_1}+\frac{s_2^2}{4g_2}\right)t$



- Single angle connected by one leg only.
  - $A_{net} = A_1 + kA_2$

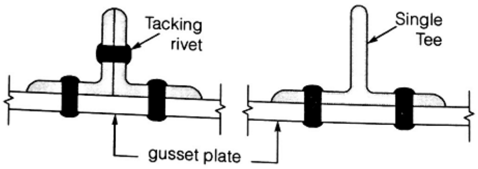


where, A_1 = Net cross-section of area of the connected leg.
A_2 = Gross cross-sectional area of unconnected leg. (out stand)



  - $k = \frac{3A_1}{3A_1 + A_2}$
  - $A_1 = \left(t_1 - \frac{t}{2}\right)t$
  - $A_2 = \left(t_2 - \frac{t}{2}\right)t$
  - $A_{net} = (I_1 + I_2 - t)t$

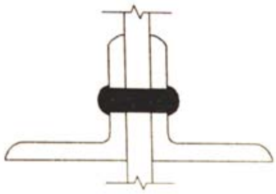
- For pair of angle placed back to back (or a signal tee) connected by only one leg of each angle (or by the flange of a tee) to the same side of a gusset plate: or it the two angles are tagged along a-a.



- $A_{net} = A_1 + kA_2$
- $k = \dfrac{5A_1}{5A_1 + A_2}$

where, A_1 = Area of connected leg
A_2 = Area of outstand (unconnected leg)

- 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 $k = \dfrac{3A_1}{3A_1 + A_2}$

Permissible Stress in Design

- The direct stress in axial tension on the effective net area should not exceeded σ_{at} where
  - $\sigma_{at} = 0.5f_y$
  - 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 Member

Strength of an Axially Loaded Compression Member

- The maximum axial compressive load P

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

where,

- P = axial compressive load (n)
- σ_{ac} = permissible stress in axial compression (MPa)
- A = gross-sectional area of the member (mm^2)
- σ_{ac} is given as

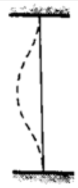
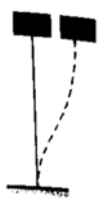
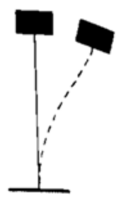
$$\sigma_{ac} = 0.6 \times \frac{f_{cc} \times f_y}{[f_{cc}^n + f_y^n]^{1/n}}$$

- f_{cc} = elastic critical stress in compression $= \dfrac{\pi^2 \times E}{\lambda^2}$

- λ = slenderness ratio = $\frac{I}{r}$

Maximum Slenderness Ratio

- A member carrying compressive loads resulting from dead load and superimposed loads has maximum slenderness ratio = 180
- 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
- A member normally carrying tension but subjected to reversal of stress due to wind or earthquake forces=350

Sl. 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	

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
- 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

Lacings

Type of lacing	**Effective length l_e**
Single lacing, riveted at ends	Length between inner and rivets on lacing bar (= I, 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 x I)
Welded lacing	0.7 times distance between inner ends of effective lengths of welds at ends (0.7 xl)

For local Buckling criteria

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

$$\not> 0.7\lambda_{whole\,section}$$

Where,

- L = distance between the centres of connections of the lattice bars to each component
- $r_{\min}^{c}$ = minimum radius of gyration of the components of compression 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 $= \frac{\text{force}}{\text{gross area}} = \frac{F}{b \times t} \not> \sigma_{ac}$
- Tensile stress in each bar $= \frac{\text{force}}{\text{net area}} = \frac{F}{(b-d) \times t} \not> \sigma_{at}$
- Numbers of rivets required $= \frac{\text{2Fcos}}{\text{Rivet value}}$

Welded connections

- **Lap joint:** Overlap $\not< (14)$ 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.

Slab Base

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

- The thickness of a square slab base plate under a solid round column.
$$t = 10\sqrt{\frac{90W}{16\sigma_{bs}} \times \frac{B}{(B-d_0)}}$$

Structural Fasteners

Riveting

- **Gross dia of rivet or dia of hole**

  d' = d + 1.5 mm for d ≤ 25 mm

  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

- **Black bolts**
  - 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**
  - 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.

Failure of Riveted/Bolted Joints

- **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

- **Strength of rivet in single shear**

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

- **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.

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

Maximum permissible stress in rivets & bolts

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

- 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) (ii) rivets lying in a line parallel to the force in the member: (a) in tension (b) in compression	 32 t or 300 mm, Whichever is less 16 t or 200 mm. whichever is less 12 t or 200 mm. whichever is less

Where t = thickness of thinner outside plate

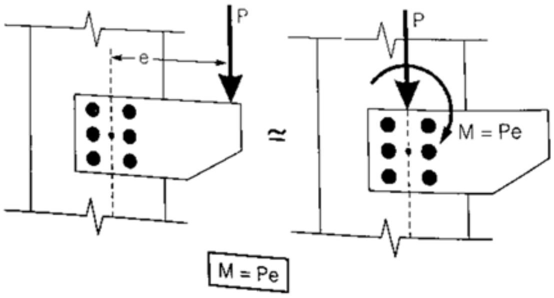
Permissible Stresses

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

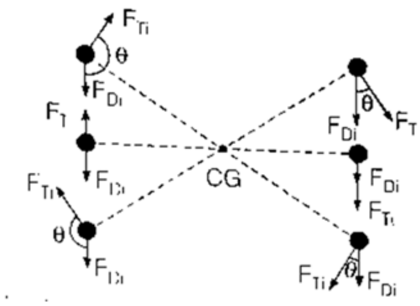
- Max permissible horizontal and vertical deflection $= \frac{\text{Span}}{325}(LSM)$
- Max permissible deflection when supported elements are susceptible to cracking $= \frac{\text{Span}}{360}(LSM)$
- Max permissible deflection when supported elements are not susceptible to cracking $= \frac{\text{Span}}{360}(LSM)$

Eccentric Connections



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

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



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

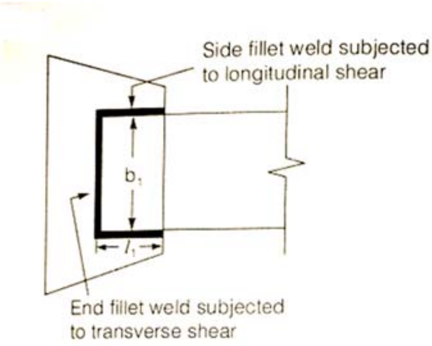
Angle b/w fusion faces	Value of k
60°-90°	0.70
91°-100°	0.65
101°-106°	0.60
107°-113°	0.55
114°-120°	0.50

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

Slide fillet weld



- $l_1 \nless b_1$
- $b_1 \ngtr 16t$ to make stress distribution uniform

if $b_1 > 16t$ use end fillet weld.

Welded Connection

- **Permissible Stresses**
- Tensions and compression on section through the throat of butt weld = 150 N/mm^2
  - Shear on section through the throat of butt of fillet weld =108 N/mm^2 ≅ 100 N/mm^2
  - Throat thickness t = k x size of weld

- **Butt-welded Joint Loaded Eccentrically**
  - Let thickness of weld throat = t, and length of weld = d
  - Shear stress at weld, $P_s = \dfrac{W}{d \times t}$

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

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

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

  - **Equivalency Method**

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

  - Permissible bending stress for flanged section = 165 N/mm^2 = 0.67f_y
  - For solid section (W, d, V) permissible bending stress is 185 N/mm^2

- **Fillet-Welded Joint Loaded Eccentrically**
  - **Load not lying in the plane of the weld:**
  - 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,

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

- For design of this connection, the depth of weld may be estimated approximately by

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

- **Load lying in the plane of the weld:**
  - Vertical shear stress at weld,

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

where,
$I(l_1 + l_2 + l_3) =$ the length of weld

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

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

  - The resultant stress,

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

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

Over 40 mm	230	151.8

• **Cantilever beams of projecting length L,**	
(a) Built-in at the support, free at end (b) Built-in at the support, restrained against torsion at the end by continuous construction. (c) Built-in at the support, restrained against lateral deflection and torsion at the free end by continuous cross members over several beams (d) Continuous and unrestrained against torsion at the support and free at the end (e) Continuous and partially restrained against torsion at the support and free at end (f) Continuous at the support, restrained against torsion at the support and free at the end	l = 0.85L l = 0.75L l =0.5L l = 3L l = 2L l = L

Effective Length of Compression Flange:

- 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

(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 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

Beam Column

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

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

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)

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

Plate Girders

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

Design of Web

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

Web stiffeners

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

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

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.

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

- Lateral loading on stiffener:

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

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,

$$I \not< \frac{D^3 \times T}{250} \times \frac{R}{W}$$

Plastic Analysis

Load Factor

- The load factor (λ)

$$\lambda = \frac{\text{Collapse load}}{\text{Service load}} = \frac{P_c}{P}$$

Saving in material

- % saving in material

$$= \left[1 - \frac{\text{Area required by plastic theory}}{\text{Area required by elastic theory}}\right] \times 100$$

Plastic Sections Modulus

Shape factors for different shapes

Section	Shape Factor (∝)
1. Rectangular section	1.5
2. (a) Triangular section (vertex upward)	2.34
(b) Triangular section (vertex horizontal)	2.00
3. Solid circular section	1.7
4. Hollow circular section (K = Ratio of inner diameter to outer diameter)	$17 \times \frac{(1-K^3)}{(1-K^4)}$

	1.27
5. Thin circular ring solid	2.00
6. (a) Diamond section (Rhombus)	1.50
(b) Thin hollow rhombus	
7. I-section	≈1.12
(a) About strong axis	≈1.55
(b) About weak axis	≈1.90 to 1.95
8. T-section	

Length of plastic hinge (L_p)

(a) For simply supported beam carrying concentrated load, length of plastic hinge is given by

$$Lp = \frac{L}{3} \text{ (for rectangular section)}$$

$$Lp = \frac{L}{8} \text{ (for I section) } Lp = L\left[1 - \frac{1}{\alpha}\right]$$

(b) For simply support beam carrying UDL length of plastic hinge is given by

$$Lp = L\sqrt{1 - \frac{1}{\alpha}}$$

Collapse Loads

- Simply supported beam with concentrated load at the center

$$w_c = \frac{4M_p}{l}$$

- Simply supported beam with uniformly distributed load

$$w_c = \frac{16M_p}{l}$$

- Propped cantilever with concentrated load at the center

$$w_c = \frac{6M_p}{l}$$

- Propped cantilever with uniformly distributed load

$$w_c = \frac{11.656M_p}{l}$$

- Fixed beam with concentrated load at the center

$$w_c = \frac{8M_p}{l}$$

- Fixed beam with eccentric loading

$$w_c = \frac{2l}{ab} M_p$$

- Fixed beam with uniformly distributed load

$$w_c = \frac{16M_p}{l^2}$$

- Fixed beam with hydrostatic loading

$$w_c = \frac{18\sqrt{3}Mp}{l^2}$$

- Continuous beam with uniformly distributed load

$$w_c = \frac{11.656Mp}{l^2}$$