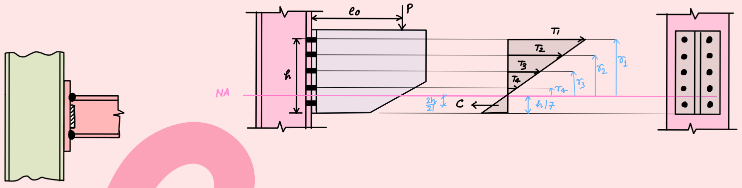

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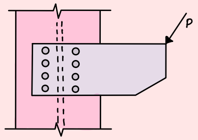
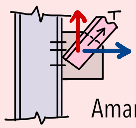


Design of Steel Structures

Prof. Chinmoy Kolay & Prof. Amar Nath R Chowdhary



TENSION MEMBERS

Design inequality for Tension Members

$$T \leq T_d$$

Factored design force (Demand) ← T; T_d → Design strength as per failure modes (Capacity)

Design strength as per various failure modes (Three Limit State)

- Gross Section Yielding, T_{dg}
- Net Section Rupture, T_{dn}
- Block Shear Failure, T_{db}

Design strength in Tension $T_d = \min(T_{dg}, T_{dn}, T_{db})$

Gross Section Yielding $T_{dg} = \dfrac{A_g f_y}{\gamma_{m0}}$ (Clause 6.2)

Net Section Rupture

$T_{dn} = \dfrac{0.9 A_n f_u}{\gamma_{m1}}$ (Clause 6.3.1)

A_n = Net cross sectional area, 0.9 is reduction factor

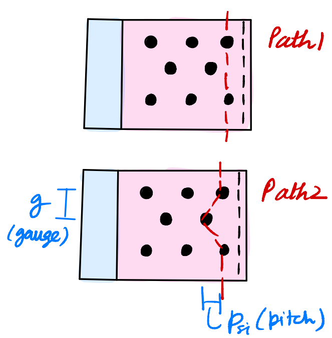
Failure Paths: $A_n = \min(A_{n1}, A_{n2})$

$A_{n1} = A_g - n d_h t$

(Gross Area − Bolt holes)

$A_{n2} = A_g - n d_h t + \left(\sum \dfrac{p_{si}^2}{4g}\right) \times t$

(Staggered Bolts)



- Special Rules apply for angles connected through one leg, T-sections and channels connected through outstands (Shear Lag Effects)

$T_{dn} = \dfrac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta \dfrac{A_{go} f_y}{\gamma_{m0}}$ (Clause 6.3.3)

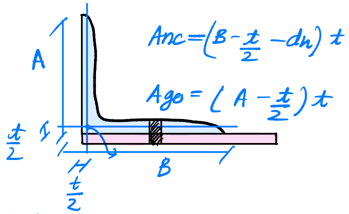
shear lag factor $\beta = 1.4 - 0.076 \left(\dfrac{w}{t}\right)\left(\dfrac{f_y}{f_u}\right)\left(\dfrac{b_s}{L_c}\right)$ (Clause 6.3.3)

$\beta_{min} = 0.7$

$\beta_{max} = 0.9 \dfrac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$

A_{nc} = Net area of connected leg

A_{go} = Gross area of outstanding leg



All distances for area measurement are taken from the center at $\frac{t}{2}$ distance from end to account for the loss in area when opening angle

Block shear Failure

Block shear and Tension at the end connection.

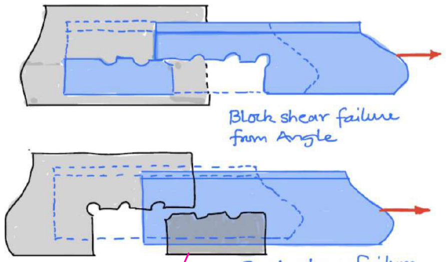
$$T_{db} = \min(T_{db1}, T_{db2})$$

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

Shear Yielding on gross area; Tension rupture on net Area

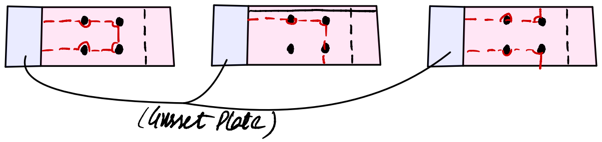
$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

Shear rupture on net area; Tension Yield on gross area



This case can happen if thickness of gusset plate is lesser than the angle / tension member thickness. (Not generally happen)

(Clause 6.4.1)



CONNECTIONS

DESIGN OF BEARING TYPE CONNECTIONS

- $V_{sb} \leq V_{db}$

  Factored shear force; Design strength of bolt

- $V_{db} = \min(V_{dsb}; V_{dpb})$ — Clause 10.3.2

  V_{dsb}: Design shear strength of bolt — Limit state of shear; V_{dpb}: Design bearing strength of bolt on any plate — Limit State of Bearing

- Each bolt is assumed to take equal share of load

  No. of bolts required $= \frac{P_u}{V_{db}}$ — P_u = factored applied load

- **Limit State of Shear in bearing type connection** — Clause 10.3.3

  $V_{dsb} \leq V_{nsb}/\gamma_{mb}$; $\gamma_{mb} = 1.25$

  $$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nsb} + n_s A_{sb}\right)\beta_{lj}\,\beta_{lg}\,\beta_{pk}$$

  n_n: shear planes in threads; n_s: shear planes in shank; $\beta_{lj}\,\beta_{lg}\,\beta_{pk}$: Reduction factors

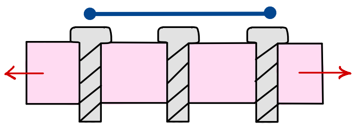
  $A_{sb} = \frac{\pi}{4}(d_b)^2$ $\qquad$ $A_{nsb} = 0.78\,A_{sb}$ (threads are intercepted)

Reduction factor for long joints — Clause 10.3.3.1

when joint length exceeds $15d$

$\beta_{lj} = 1.075 - 0.005(l_j/d)$; $0.075 \leq \beta_{lj} \leq 1.0$

Not applied when shear distribution is uniform (eg. connection of web to flange)

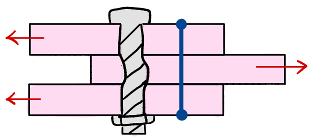


Reduction factor for long grips — Clause 10.3.3.2

When joint grip exceeds $5d$ but restricted to max $8d$

$\beta_{lg} = 8/(3 + l_g/d) \leq \beta_{lj}$

As grip length ↑es, bolts are subjected to greater bending moments in addition to shear forces

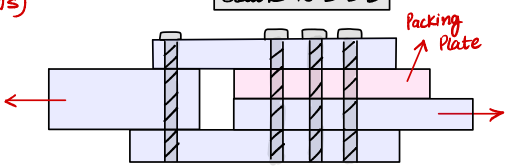


Reduction factor for packing plates (shims) — Clause 10.3.3.3

When shims are thicker than 6mm

$\beta_{pk} = 1 - 0.0125\,t_{pk}$

t_{pk} = thickness of thicker packing (mm)



NO reductions ($\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$)

- **Limit State of Bearing in bearing type connection** — Clause 10.3.4

  $V_{dpb} \leq V_{npb}/\gamma_{mb}$; $\gamma_{mb} = 1.25$

  $$V_{npb} = (2.5\,k_b\,d\,t\,f_u)\,\beta_{lr}$$

  $$k_b = \min\left(\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1\right)$$

  $$\beta_{lr} = \begin{cases} 1.0 & \text{STD} \\ 0.7 & \text{OVS \& SSL} \\ 0.5 & \text{LSL} \end{cases}$$

  f_{ub} = UTS of bolt $\quad d = d_b$

  f_u = UTS of plate $\quad d_0 = d_n$

  $t = \min(\Sigma t$ experiencing bearing stress in same direction$)$

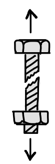
- **Limit State of Tension in bearing type connection** — Clause 10.3.5

  $T_b \leq T_{db}$

  $T_{db} = T_{nb}/\gamma_{mb}$; $\gamma_{mb} = 1.25$

  $$T_{nb} = \min\left(0.9 f_u A_{nsb} ;\ f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{m0}}\right)$$

  $0.9 f_u A_{nsb}$: Rupture strength on net bolt area at bottom; $f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{m0}}$: Yield strength on shank area ↑ed by γ_{mb}



- **Block shear in bearing type connection**

  Clause 6.4 as done in tension members

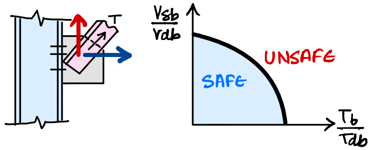
- Combined Shear and Tension in bearing type connection

$\boxed{\text{Clause } 10.3.6}$

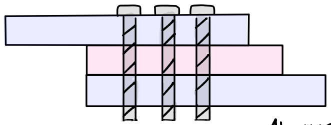
$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

$\frac{V_{sb}}{V_{db}}$: Ratio of factored shear force demand to design shear strength (Clause 10.3.2)

$\frac{T_b}{T_{db}}$: Ratio of factored tensile force demand to design tensile strength (Clause 10.3.5)



A conservative estimate for n_n and n_s



Assume

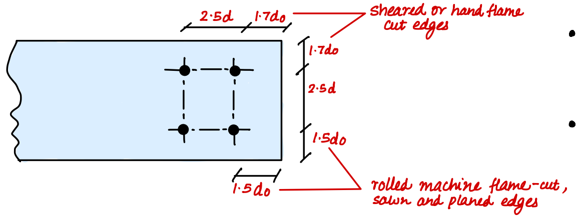
n_n = no. of shear plane thro' threads = 2

n_s = " " " " shank = 0

Always assume that threads pass through shear plane

SPACING FOR FASTENERS

MINIMUM SPACING FOR FASTENERS (Clause 10.2.2 & 10.2.4)



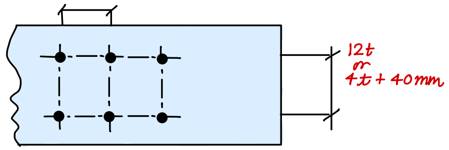
- Minimum spacing b/w bolts

$s = \min(p, g)$, $d_{min} = 2.5d_o$ } $s \geq d_{min}$

- Minimum edge and end distance

$e_{min} = 1.5d_o$ S,HF ; $1.7d_o$ RMF } $e \geq e_{min}$

MAXIMUM SPACING FOR FASTENERS



Minimum Spacing between bolts

$d_{min} = 2.5d$

$s_{min} = \min(p, g) \geq d_{min}$

Maximum spacing between bolts

$d_{max} = \min(32t, 300\text{mm})$

$s_{max} = \sqrt{p^2 + g^2} \leq d_{max}$

Maximum pitch distance b/w bolts

$$P_{max} = \begin{cases} \min(16t, 200\text{mm}) & \text{in tension members} \\ \min(12t, 200\text{mm}) & \text{in compression members} \end{cases}$$

$p \leq p_{max}$

Maximum spacing of fasteners in a line adjacent and parallel to the edge of the outside plate.

$d_{max} = \min(4t + 100, 200)$ mm

$s_{max} = \max(p, g) \leq d_{max}$

Minimum end/edge distance

$$e_{min} = \begin{cases} 1.7d_h & \text{shear/hand flame cut edges} \\ 1.5d_h & \text{rolled, machine flame cut, sawn and planed edges} \end{cases}$$

$e \geq e_{min}$

Maximum end/edge distance

$e_{max} = 12t\sqrt{\frac{250\text{MPa}}{f_y}}$

t → thickness of thinner plate.

$e \leq e_{max}$

FRICTION GRIP TYPE BOLTING (SLIP CRITICAL CONNECTIONS)

- Slip Resistance

$V_{sf} \leq V_{dsf}$

V_{sf}: Factored design force at which no slip takes place at the interface (Demand)

V_{dsf}: Design shear capacity governed by slip for friction grip connection (Capacity)

$\boxed{\text{Clause } 10.4.3}$

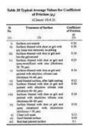
$V_{dsf} \leq V_{nsf}/\gamma_{mf}$

$\boxed{\text{Clause } 10.4.3}$

$$\gamma_{mf} = \begin{cases} 1.25 & \text{slip resistance at ultimate loads} \\ 1.10 & \text{" " at working loads} \end{cases}$$

$$V_{nsf} = \mu_f n_e K_h F_o \beta_{lj}$$

μ_f: Slip Factor [Table 20]



n_e: no. of effective frictional surface

K_h: Factor for hole size

β_{lj}: factor for long joint (Clause 10.3.3.1)

F_o: Minimum bolt tension (proof load)

$F_o = A_{nsb} f_o$

f_o = proof stress = $0.7 f_{ub}$

Factor for hole size

$$K_h = \begin{cases} 1.00 & \text{STD} \\ 0.85 & \text{OVS, SSL \& LSL loaded} \perp \text{to slot} \\ 0.70 & \text{LSL loaded} \parallel \text{to slot} \end{cases}$$

Slip Factor (Table 20)

Proof load

$F_o = A_{nsb} f_o = A_{nsb}(0.7 f_{ub})$

F_o: Proof load; f_o: Proof stress; f_{ub}: Bolt UTS

- **Tension Resistance**

$T_f \leq T_{df}$

Factored tension force

Design tensile capacity

Clause 10·4·5

$T_{df} \leq T_{nf} / \gamma_{mf}$

$T_{nf} = \min \left(0.9 f_{ub} A_{nb} ;\ f_{yb} A_{sb} \frac{\gamma_{mf}}{\gamma_{mo}} \right)$

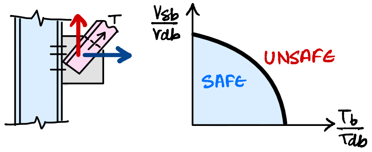
- **Combined shear and Tension**

Clause 10·4·6

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

Ratio of factored shear force demand to slip resistance (Clause 10.4.3)

Ratio of factored tension force demand to design tensile strength (Clause 10·4·5)



SIMPLE WELDED CONNECTIONS

(WELDING)

Choosing an electrode

Table 1 Tensile Properties of Structural Steel Products

(*Clauses* 1.3.113, 1.3.119 *and* 2.2.4.2)

Sl No.	Indian Standard	Grade/Classification	Properties: Yield Stress MPa, *Min*	Ultimate Tensile Stress MPa, *Min*	Elongation, Percent, *Min*
(1)	(2)	(3)	(4)	(5)	(6)
		Ex40xx	330	410-540	16
		Ex41xx	330	410-540	20
		Ex42xx	330	410-540	22
		Ex43xx	330	410-540	24
		Ex44xx	330	410-540	24
ii)	IS 814	Ex50xx	360	510-610	16
		Ex51xx	360	510-610	18
		Ex52xx	360	510-610	18
		Ex53xx	360	510-610	20
		Ex54xx	360	510-610	20
		Ex55xx	360	510-610	20
		Ex56xx	360	510-610	20

WELD SIZES

FILLET WELD

Max size — **Clause 10·5·8**

- $S_{max} = t - 1.5$ mm for square edges of $t > 6$mm
- $S_{max} = t$ for square edges of $t < 6$mm
- $S_{max} = 0.75t$ for the rounded edges of rolled sections

Max size — **Clause 10·5·8·5**

End filled weld normal to force direction

Throat thickness not less than $0.5t$

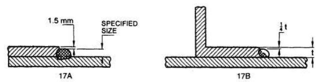


Fig. 17 Fillet Welds on Square Edge of Plate or Round Toe of Rolled Section

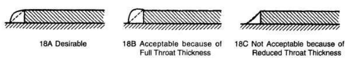
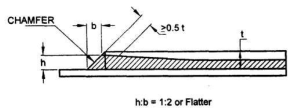


Fig. 18 Full Size Fillet Weld Applied to the Edge of a Plate or Section



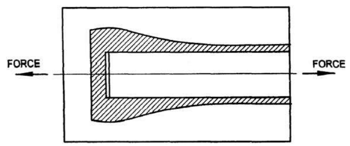


Fig. 19 End Fillet Weld Normal to Direction of Force

Min size

- $S_{min} = 3$ mm for $t_{max} \leq 10$mm
- $S_{min} = 5$mm " 10mm $\leq t_{max} \leq 20$mm
- $S_{min} = 6$mm " 20mm $\leq t_{max} \leq 32$mm
- $S_{min} = 8$mm for the first run and 10mm for 32 mm $\leq t_{max} \leq 40$mm

Clause 10·5·2·3 Table 21

Table 21 Minimum Size of First Run or of a Single Run Fillet Weld

(*Clause* 10.5.2.3)

Sl No.	Thickness of Thicker Part mm: Over	Up to and Including	Minimum Size mm
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

2 Where the thicker part is more than 50 mm thick, special precautions like pre-heating should be taken.

BUTT WELDS

- Min groove depths for different situations applicable
- End returns: min of 2 times weld size
- Min length $L_{min} = \max(4s, 40\text{mm})$
- Lap joints: min lap length $L_{lap} = 4t$ mm or 40 mm

Stresses in Fillet welds

— Due to individual forces

$f_a = N / (l_w t_t)$ Axial force

$q = Q / (l_w t_t)$ Shear force Clause 10·5·9

— Due to combination of stresses Clause 10·5·10

$f_e = \sqrt{f_a^2 + 3q^2} \leqslant \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$ Clause 10·5·10·1·1

Stresses in Butt welds

— Due to combination of stresses Clause 10·5·10

Combined shear and bending

$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$ Clause 10·5·10·1·1

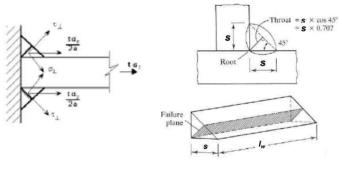
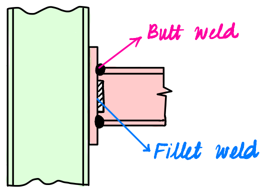


Table 22 Values of *K* for Different Angles Between Fusion Faces
(*Clause* 10.5.3.2)

Angle Between Fusion Faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
Constant, *K*	0.70	0.65	0.60	0.55	0.50



Design of fillet weld connection Clause 10·5·7·1·1

— Design strength

$R_{wdf} = (l_w t_t) f_{wd} \beta_{lw}$

Effective throat area

Reduction factor for long joints

$\beta_{lw} = 1{\cdot}2 - \frac{0{\cdot}2\, l_j}{150\, t_t} \leqslant 1{\cdot}0$ Clause 10·5·7·3

Design stress

$f_{wd} = f_{wn} / \gamma_{mw}$

$\gamma_{mw} = \begin{cases} 1{\cdot}25 & \text{Shop welds} \\ 1{\cdot}50 & \text{Field welds} \end{cases}$

$f_{wn} = \frac{f_u}{\sqrt{3}}$; $f_u = \min(f_{uw}, f_{up})$

Design of butt weld connection Clause 10·5·7·1·2

$R_{wdb} = (l_w t_t) f_{wd} \beta_{lw}$

Effective throat area with throat thickness equal to thickness of plate.

Design stress

$f_{wd} = f_{wn} / \gamma_{mw}$

$\gamma_{mw} = \begin{cases} 1{\cdot}25 & \text{Shop welds} \\ 1{\cdot}50 & \text{Field welds} \end{cases}$

$f_{wn} = \frac{f_u}{\sqrt{3}}$; $f_u = \min(f_{uw}, f_{up})$

DESIGN OF WELD (FILLET WELD)

Min. weld size $s_{min} =$ (Table 21, IS800)

Max. weld size $s_{max} =$ (Figure 18, IS800)

Effective throat thickness factor, $K =$ (Table 22, IS800)

Nominal weld strength $f_{wn} = f_{uw} / \sqrt{3}$

Design weld strength $f_{wd} = f_{wn} / \gamma_{mw}$

Strength of weld per unit length $F_w = f_{wd} \cdot K \cdot s$

Required length of weld $L_w = \frac{P}{F_w}$ ⤳ factored external force

Provide weld length = roundoff $(L_w, 2)$

Bolts

Table 1 Tensile Properties of Structural Steel Products
(*Clauses* 1.3.113, 1.3.119 *and* 2.2.4.2)

Sl No.	Indian Standard	Grade/Classification	Properties: Yield Stress MPa, *Min*	Ultimate Tensile Stress MPa, *Min*	Elongation, Percent, *Min*
(1)	(2)	(3)	(4)	(5)	(6)
iv)	IS 1367 (Part 3)	3.6	—	330	25
		4.6	—	400	22
		4.8	—	420	—
		5.6	—	500	20
		5.8	—	520	—
		6.8	—	600	—
		8.8 ($d \leq$ 16 mm)	640 [1)]	800	12
		8.8 ($d >$ 16 mm)	660 [1)]	830	12
		9.8	720 [1)]	900	10
		10.9	940 [1)]	1 040	9
		12.9	1 100 [1)]	1 220	8

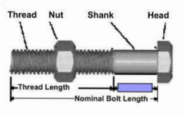
4·6 Grade Bolt

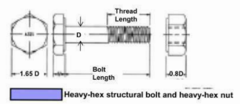
$f_{ub} = 400$ MPa

$f_{yb} = 0{\cdot}6 f_{ub} = 240$ MPa

Clearance for fasteners (bolts)

↳ Table 19, IS800





COMPRESSION MEMBERS

Limit States for Compression Members

- Flexural Buckling
- Torsional Buckling
- Flexural Torsional Buckling
- Local Buckling
- Squashing

LIMIT STATE OF STRENGTH

$P \leqslant P_d$ Clause 7·1·2·1

$P_d = A_e f_{cd}$

$f_{cd} = \chi f_y / \gamma_{mo} \leqslant f_y / \gamma_{mo}$

$\chi = \frac{1}{[\phi + (\phi^2 - \lambda^2)^{0.5}]}$

$\phi = 0{\cdot}5\,[1 + \alpha(\lambda - 0{\cdot}2) + \lambda^2]$

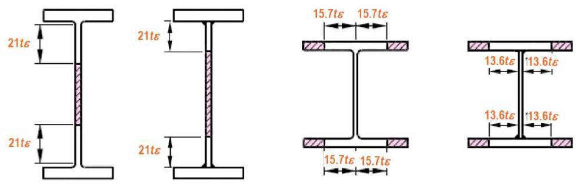
$\lambda = \sqrt{f_y / f_{cc}} = \sqrt{f_y (KL/r)^2 / \pi^2 E}$

- IS800 Approach
  Same as Perry Robertson approach
- Imperfection factor, α Table 7
- Design compressive stress f_{cd} Table 9a,b,c,d
- Section classification Table 2
- Buckling class Table 10
- Slenderness ratio limit Table 3

- Section class 1, 2 and 3 — Not slender — not affected by the local buckling i.e. $A_e = A_g$
- Section class 4 — Slender — based on reduced area (effective area)

Effective Area for Class 4 sections

- By deducting width of the compression plate element in excess of the semi-compact section (Class 3) limit



Rolled I-Section Welded I-Section Rolled H-Section Welded H-Section

FLEXURAL TORSIONAL BUCKLING

$$\lambda_{vv} = \frac{L/r_v}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \; ; \; \lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

$\boxed{\text{Clause 7.5.1.2}}$

Equivalent slenderness Ratio $\lambda_e = \sqrt{K_1 + K_2\lambda_{vv}^2 + K_3\lambda_\phi^2}$

$\boxed{\text{Table 12}}$ K_1, K_2, K_3

Equivalent Slenderness Ratio $\left(\frac{KL}{r}\right)_{eq} = \pi\lambda_e \cdot \sqrt{\frac{E}{f_y}}$

Design axial strength $P_d = f_{cd} A_e$
↳ f_{cd} from Table 9 or eq^ns

FLEXURAL MEMBERS

YIELD MOMENT

$y_{cg} = \frac{\sum A_i y_i}{\sum y_i}$, I_{zz}

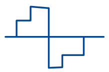
$Z_{elT} = \frac{I_{zz}}{D - y_{cg}}$, $Z_{elB} = \frac{I_{zz}}{y_{cg}}$ (Elastic Section Modulus)

$M_y = f_y Z_{el}$

$M_y = \min(Z_{elT} \cdot f_{yft}, Z_{elB} \cdot f_{yfb})$

PLASTIC MOMENT

Plastic N.A., $y_p \Rightarrow$ By solving $C = T$ (compression = tension)



M_p = By taking moment about plastic N.A.

$M_p = f_y Z_p$

Shape Factor $= \frac{M_p}{M_y} = \frac{Z_p}{Z_e}$

$M_y = \sigma_y Z_e$ $Z_e = I_{zz}/y_{max}$ (elastic modulus)
$M_p = \sigma_y Z_p$ ↳ plastic section modulus

SHEAR

$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v f_y}{\gamma_{m0}\sqrt{3}}$ $\boxed{\text{Clause 8.4}}$

SHEAR BUCKLING STRENTH

$\frac{d}{t_w} \not< 67 \cdot \varepsilon_w$ → WEB IS NOT SUSCEPTIBLE TO SHEAR BUCKLING $\boxed{\text{Clause 8.2.1.1}}$

$V_{cr} = A_v \tau_b$ → Three conditions for it based on λ_w

$\lambda_w = \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})}$ → $\tau_{cr,e} = K_v \pi^2 E / [12(1-\mu^2)(d/t_w)^2]$

K_v: 5.35 or other.... ↳ $\mu = 0.3$

MOMENT CAPACITY

$M_d = \frac{\beta_b M_p}{\gamma_{m0}} = \frac{\beta_b (Z_p f_y)}{\gamma_{m0}}$ $\boxed{\text{Clause 8.2.1.2}}$ For shear < 0.6 V_d

M_d = given in code → $\boxed{\text{Clause 8.2.1.3 and 9.2}}$ For shear > 0.6 V_d

STIFFENER DESIGN

Load carrying Stiffeners

- Check for Web buckling due to reaction or concentrated force $\boxed{\text{Clause 8.7.3.1}}$
- Check for Web bearing due to reaction or concentrated force $\boxed{\text{Clause 8.7.4}}$

For Plastic Hinge Problems

- Find no. of PH needed and BMD (approx.)
- Apply Principle of Virtual Displacement

$$\boxed{\Sigma P\Delta = \Sigma M_p \theta} \qquad (\theta = \Delta / L)$$

- Plastic collapse Load = min (all such P's from different mechanisms)

LATERALLY UNRESTRAINED BEAMS

ELASTIC LATERAL TORSIONAL BUCKLING (LTB) MOMENT

$$M_{cr} = \sqrt{\left(\frac{\pi^2 E I_y}{L_{LT}^2}\right)\left(G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right)} \qquad \boxed{\text{Clause } 8.2.2.1}$$

I_t = torsional constant = $\Sigma\left(\frac{b_i t_i^3}{3}\right)$

I_w = warping constant = $\frac{I_y h^2}{4}$

$KL = L_{LT}$ = effective length for LTB

Rewrite critical moment in terms of slenderness parameter (L/r_y) and (h/t) for I-sec.

$$M_{cr} = \frac{\pi^2 E I_y h_f}{2 L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} \qquad \boxed{\text{Clause } 8.2.2.1}$$

$t_w = \frac{2}{3} t_f$; $2b_f = d$

$A = 2b_f t_f + d t_w$

$I_t = \frac{1}{3} b_f t_f^3 \times 2 + \frac{1}{3} d t_w^3$

$G = \frac{E}{2(1+\nu)} = \frac{E}{2.5} = 0.4E$

$I_t = 0.27 t_f^2 A$

$I_w = \frac{I_y h^2}{4} \approx 0.226 A r_y^2 d^2$

$h_f = 0.98d = d - t_f$

$h = h_f$ c/c between flanges

Elastic Critical Buckling Stress in terms of slenderness parameter (L/r_y) and (D/t) for I sec.

elastic lateral buckling stress

$f_{cr,b} = \frac{M_{cr}}{Z_p \beta_b}$

$= \frac{M_{cr}}{Z_e}$ for semi-compact section

$\beta_b = 1.0$ for plastic and compact section

$= Z_e/Z_p$ for semi-compact section

$$f_{cr,b} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} \qquad \boxed{\text{Clause } 8.2.2.1}$$

Critical Buckling Moment for uniform bending moment diagram

$$M_{cr} = \sqrt{\left\{\underbrace{\left(\frac{\pi^2 E I_y}{L_{LT}^2}\right)}_{\text{Lateral Flexure Buckling}}\underbrace{\left[G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right]}_{\text{Torsional Effect (Pure Torsion + Warping)}}\right\}} \qquad \boxed{\text{Clause } 8.2.2.1}$$

DESIGN BUCKLING STRENGTH

Plastic (or yield) moment capacity reduced for LTB

$M_d = \chi_{LT} \beta_b M_p / \gamma_{m0}$

$= \chi_{LT} \beta_b Z_p f_y / \gamma_{m0}$

$$\chi_{LT} = \frac{1}{\{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}\}} \leq 1.0$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 Z_e f_y / M_{cr}}$$

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{L_{LT}^2}\right)\left[G I_t + \frac{\pi^2 E I_w}{L_{LT}^2}\right]\right\}} \qquad \boxed{\text{Clause } 8.2.2}$$

BEAM COLUMN

SECTION STRENGTH

Clause 9.3.1

Class 1 (Plastic) and class 2 (compact) cross sections

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leqslant 1.0$$

Plastic and compact section

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leqslant 1.0$$

Conservative Approach

Class 3 (semi compact) cross sections

$$f_x \leq f_y / \gamma_{mo}$$

Semi-compact section

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leqslant 1.0$$

Conservative Approach

$f_x = \sigma_c + \sigma_b$ (combined axial and bending stress)

OVERALL MEMBER STRENGTH

Clause 9.3.2.2

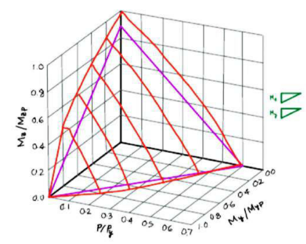
$$\frac{P}{P_{dy}} + K_y \frac{C_{my} M_y}{M_{dy}} + K_{LT} \frac{C_{mz} M_z}{M_{dz}} \leqslant 1.0$$

Buckling about weak axis + Bending about weak axis / minor axis + LTB

$$\frac{P}{P_{dy}} + 0.6 K_y \frac{C_{my} M_y}{M_{dy}} + K_z \frac{C_{mz} M_z}{M_{dz}} \leqslant 1.0$$

Buckling about major axis

Biaxial Bending



K_y, K_z, K_{LT} : Interaction factors or magnification factors

C_{my}, C_{mz} : Equivalent uniform moment factors

NOTES ON SECTION CLASSIFICATION

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4)

Compression Element (1)			Ratio (2)	Class of Section: Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\geqslant 42\varepsilon_w$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\geqslant 42\varepsilon_w$
		If r_1 is positive:	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\geqslant 42\varepsilon_w$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section			d/t	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross-section.

2 $\varepsilon = (250/f_y)^{1/2}$.

3 Webs shall be checked for shear buckling in accordance with **8.4.2** when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Fig. 2, **3.7.3** and **3.7.4**).

4 Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.

5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overll section}}$$

VAGUE AND INCORRECT

BS 5950 Part 1 : 2000

3.5.5 Stress ratios for classification

The stress ratios r_1 and r_2 used in Table 11 and Table 12 should be determined from the following:

a) for I- or H-sections with equal flanges:

$$r_1 = \frac{F_c}{d t p_{yw}} \text{ but } -1 < r_1 \leq 1$$

$$r_2 = \frac{F_c}{A_g p_{yw}}$$

$$r_1 = \frac{P_u}{(dt) f_{yw}} \qquad r_2 = \frac{P_u}{A_g f_{yw}}$$

Clause 8.2 | Clause 9.2.2

IF $V_{uy} < 0.6 V_{dy}$

YES → Section class $\geqslant 3$

- YES → $\beta_b = Z_{ez} / Z_{pz}$
- NO → $\beta_b = 1$

$$M_{dz} = Z_p \beta_b \frac{f_y}{\gamma_{mo}}$$

$Z_p \to Z_{pz}$

NO → Section class $\geqslant 3$

- YES → $M_{dz} = \frac{Z_{ez} f_y}{\gamma_{mo}}$
- NO → $M_{fd} = \frac{f_{yw}}{\gamma_{mo}} (d + t_f) B$

$$\beta_b = \left(2 \frac{V}{V_d} - 1\right)^2$$

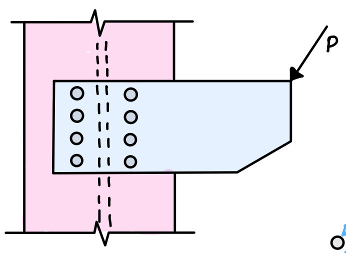
$$M_d = Z_{pz} f_y$$

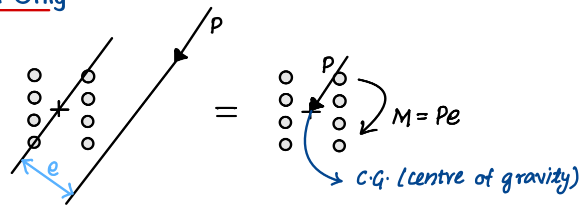
$$M_{dz} = M_{dv} = \max\left(M_d - \beta(M_d - M_{fd}), \frac{1.2 Z_{ez} f_y}{\gamma_{mo}}\right)$$

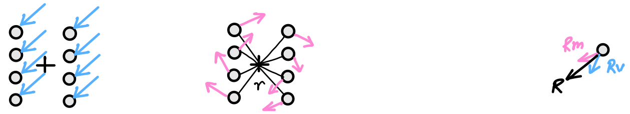
ECCENTRIC CONNECTIONS

Definition: Resultant of applied forces does not pass through the C.G. of $\frac{\text{fastener}}{\text{weld}}$ group.
Two Types: • Cause only shear in fasteners/weld
• Cause shear + tension in fasteners/weld

Eccentric Bolted Connection: Shear Only







Direct shear $R_v = \frac{P}{N}$ Torsional shear $R_m = \frac{Mr}{\Sigma r^2}$ Total shear R

Bolts are subjected to > direct shear due to P
> torsional shear due to torque $M = Pe$

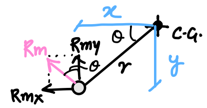
Assuming fastener group to be a c/s subjected to torsional moment $M = Pe$

The torsional shear stress $f_v = \frac{Mr}{J}$, $J = \Sigma Ar^2$ (Ignore J about own axis of bolts)

$f_v = \frac{Mr}{\Sigma Ar^2}$

Shear Force due to moment $R_m = f_v \cdot A = \frac{Mr}{\Sigma r^2}$

Elastic Analysis



$r^2 = x^2 + y^2$

$\Sigma r^2 = \Sigma(x^2 + y^2)$

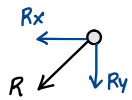
$R_{mx} = R_m \sin\theta = \frac{Mr\sin\theta}{\Sigma r^2} = \frac{My}{\Sigma(x^2+y^2)}$

$R_{my} = R_m \cos\theta = \frac{Mx}{\Sigma(x^2+y^2)}$

$R_x = R_{vx} + R_{mx}$

$R_y = R_{vy} + R_{my}$

$R = \sqrt{R_x^2 + R_y^2}$



Alternate Analysis

Clause 10.11.1.1(c)

Let the shear force due to torsional moment be R_m

$R_m \propto r$

$R_m = pr$, p = const.

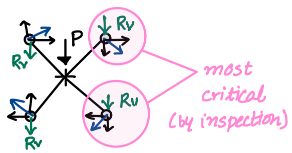
Equilibrium $\Rightarrow \Sigma R_m \times r = M$

$\Rightarrow \Sigma pr^2 = M$

$p = \frac{M}{\Sigma r^2}$

Thus, $R_m = \frac{Mr}{\Sigma r^2}$

NOTE: Most critical bolt is the one which is farthest from C.G. and in which the direct shear and torsional shear forces add up.



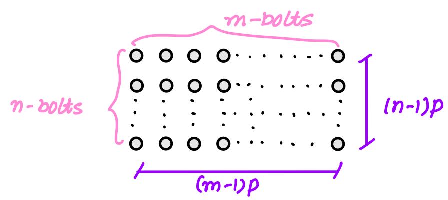
DETERMINATION OF NO. OF BOLTS

For a vertical row of bolts

$\Sigma y^2 = \frac{p^2 n(n-1)(n+1)}{12}$

For m vertical rows

$\Sigma y^2 = \left[\frac{p^2 n(n-1)(n+1)}{12}\right] \times m$



Similarly, $\Sigma x^2 = \left[\frac{p^2 m(m-1)(m+1)}{12}\right] \times n$

$\Sigma r^2 = \Sigma(x^2+y^2) = \frac{p^2 mn(m^2+n^2-2)}{12}$

For the extreme bolts, $r = \sqrt{\left(\frac{n-1}{2}\cdot p\right)^2 + \left(\frac{m-1}{2}\cdot p\right)^2} = \frac{p}{2}\sqrt{(n-1)^2+(m-1)^2}$

Let us ignore the direct shear and equate $R_m = \frac{Mr}{\Sigma r^2}$ with bolt value R,

$R_m = \frac{Mr}{\Sigma r^2} = \frac{M\cdot(p/2)\cdot\sqrt{(m-1)^2+(n-1)^2}}{p^2 mn(m^2+n^2-2)/12} = R$ (Bolt Value)

$\frac{6M}{pmn} \times \frac{\sqrt{(m-1)^2+(n-1)^2}}{(m^2+n^2-2)} = R \Rightarrow R \approx \frac{6M}{pmn^2} \Rightarrow n = \sqrt{\frac{6M}{pmR}}$

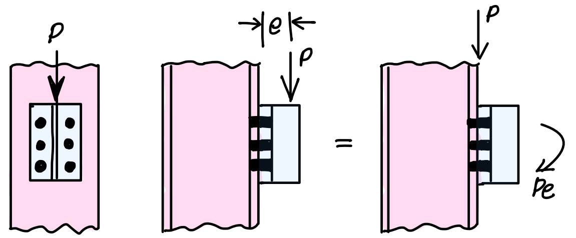
if $n \gg m$ $\approx 1/n$

n = no. of bolts
m = no. of vertical rows

Aman Kumar Singh \ IITK

Eccentric Bolted Connection: Shear and Tension

Bolts are subjected to > direct shear due to P
> tension due to moment $M = Pe$

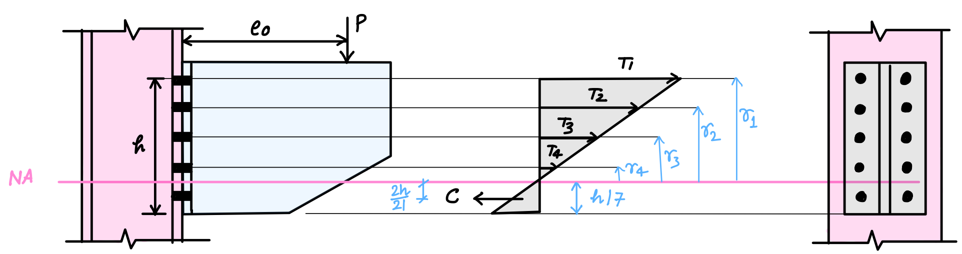


ANALYSIS PROCEDURE

1) Calculate shear in each bolt $V_{sb} = \frac{P}{N}$ (N = no. of bolts)
2) Calculate Bolt Tension T_b
3) Check for Shear Moment Interaction

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

Clause 10.3.6



N.A. is typically assumed at $\frac{h}{7}$ from the bottom of bracket (usually at $h/6$ to $h/7$ but $h/7$ is more conservative due to smaller value)

By similar Δs, $\frac{T_1}{r_1} = \frac{T_2}{r_2} = \frac{T_3}{r_3} = \cdots\cdots$

Moment taken by bolts in Tension $M' = \Sigma T_i r_i = T_1 r_1 + T_1 \frac{r_2^2}{r_1} + T_1 \frac{r_3^2}{r_1} + \cdots\cdots$

$M' r_1 = T_1 \Sigma r_i^2 \Rightarrow T_1 = \frac{M' r_1}{\Sigma r_i^2}$

Similarly for i-th bolt $T_i = \frac{M' r_i}{\Sigma r_i^2}$

Now, force equilibrium $T_1 + T_2 + T_3 + \cdots = C \Rightarrow C = \frac{M' \Sigma r_i}{\Sigma r_i^2}$
or $\Sigma T_i = C$

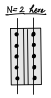
Moment of compressive force about N.A. $= C \cdot \frac{2h}{21} = \frac{2h}{21} \times \frac{M' \Sigma r_i}{\Sigma r_i^2}$

Total Moment $M = Pe_0 = M' + C \cdot \frac{2h}{21}$

$$M = M' + \frac{M' \Sigma r_i}{\Sigma r_i^2} \times \frac{2h}{21}$$

$$M' = \frac{M}{\left(1 + \frac{2h}{21} \times \frac{\Sigma r_i}{\Sigma r_i^2}\right)}$$

Max. Tension $T_1 = \frac{M' r_1}{\Sigma r_i^2} = \frac{M r_1}{N\left(\Sigma r_i^2 + \frac{2h}{21} \Sigma r_i\right)}$ → for N vertical rows of bolt



Interaction Equation

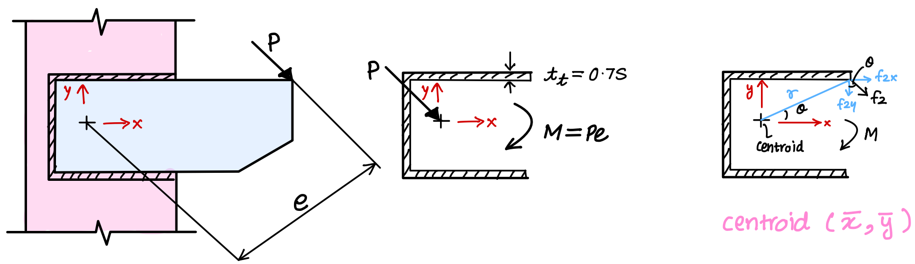
$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

Clause 10.3.6

- AISC Case-I
- AISC Case-II

} Other two approaches!

Eccentric Welded Connection: Shear Only



Assume unit throat thickness ($t_t = 1$) for simplification

Direct shear stress due to P, $f_1 = \frac{P}{L}$

$f_{1x} = \frac{P_x}{L}$ $f_{1y} = \frac{P_y}{L}$ L = total length of weld

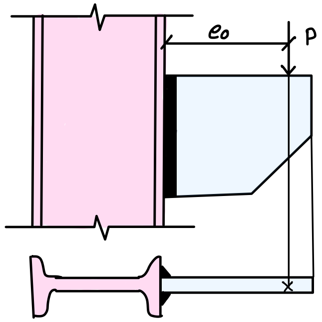
Shear stress due to torsional moment, $f_2 = \frac{Mr}{J}$

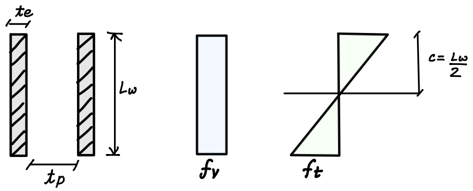
$f_{2x} = \frac{Mr\sin\theta}{J} = \frac{My}{J}$ $f_{2y} = \frac{Mr\cos\theta}{J} = \frac{Mx}{J}$

$J = \int_A r^2 dA = \int_A (x^2 + y^2) dA = I_x + I_y$

Total shear stress $f_v = \sqrt{(\Sigma f_x)^2 + (\Sigma f_y)^2} \leq f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}}$

Eccentric Welded Connection: Shear and Tension





$$C = \frac{L_w}{2}$$

$$I = \frac{2 \cdot t_e L_w^3}{12} = \frac{t_e L_w^3}{6}$$

$$Z = \frac{I}{C} = \frac{t_e L_w^2}{3}$$

Direct shear stress $f_v = \frac{P}{A} = \frac{P}{2 L_w t_e}$

Max. tensile stress $f_t = \frac{Mc}{I} = \frac{3M}{t_e L_w^2}$

Resultant $f_r = \sqrt{f_v^2 + f_t^2} \leq f_{wd} = \frac{f_u}{\sqrt{3}\, \gamma_{mw}}$