Steel Design Flowcharts

Updated 12/9/16

Still Need to Make Slides for:

Combined Loading W/ First & Second order Analysis pg 350 McCor.

READ CHAPTER 2 of AISC for general INFORMATION

Beam End Bearing (Revisit) (pg 335 Textbook)

Bearing on Masonry ETC…\

Shear flow??

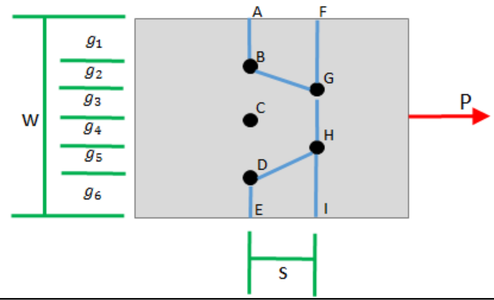
COLUMN SPLICES

Cover Plate Design w/ AISC E6 for Built up Sections/Columns & Bolts & Welds (pg 178 textbook)

AISC 14th Example II.1-20 & 21

STEEL TENSION MEMBER DESIGN

General Tensile Design Information



Tensile Design Information

- AISC Spec. D1 – Slenderness Limits
- AISC Spec. D2 – Tensile Strength
- AISC Spec. D3 – Effective Net Area (Shear Lag Table D3.1)
- AISC Spec. D4 – Built-Up Members
- AISC Spec. D5 – Pin-Connected Members
- AISC Spec. D6 – Eyebars

Important Tables

- Table D3.1 – Shear Lag Coefficients
- AISC Table 5-1 to 5-8 Available Strength in Axial Tension for Shapes
    - 5-1 (W-Shapes), 5-2 (Angles), 5-3 (WT), 5-4 (Rectangle HSS)
    - 5-5 (Square HSS), 5-6 (Round HSS) 5-7 (Pipe), 5-8 (Dbl Angle)

General Provisions

- Preferable for $\frac{L}{r} \leq 300$ ➔ r = $\sqrt{\frac{I}{A}}$
- Limit States
    - Tensile Yield $P_n = F_y A_g$ $\emptyset = 0.9$ Lowest governs
    - Tensile Rupture $P_n = F_u A_e$ $\emptyset = 0.75$
    - Ductile Failure if $0.9F_y A_g \leq 0.75F_u A_e$

Net Area

- This will take into account the effect of bolt holes
    - $A_n = A_g - A_h$
        - $A_h = n_{holes} t(d + 0.125)$ ➔ $A_n = A_g - n_{holes}(d + 0.125)t$
- Net Area for a Chain of Holes
    - $A_n = A_g - \sum d_{hole} t + \sum\left(\frac{s^2}{4g}\right) t$
    - Example Above w/ 3/4" bolts (path A-B-G-H-D-E)
        - Always use path with smallest A_n because it is weakest & governs
        - $A_n = A_g - 4(d_{bolt} + 0.125)(t_{plate}) + \left(\frac{s^2}{4g_2}\right) t + \left(\frac{s^2}{4g_5}\right) t$
- Shear Lag Coefficient (U) ➔ AISC Table D3.1 ➔ Use highest applicable U
    - Bolted connections $A_e = A_n U$
        - AISC J4.1 connecting element w/ bolts A_n must be less than $0.85A_g$
    - Welded connections $A_e = A_g U$ ➔ Case 4 is quite common. Case 2 w/ angles
    - *Hint: When using Table D3.1 Case 2 for Wide Flanges, get $\bar{x}$ from a WT section that is equivalent to half the full section (e.g. WT5x22.5 for a W10x45).
- Block Shear

  $R_n = lowest\ of\ 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$ [AISC EQ J4-5]

  U_{bs} = 1 when tension stress is uniform, 0.85 otherwise. OR 0. 5 in coped beams w/ more than 1 row bolts

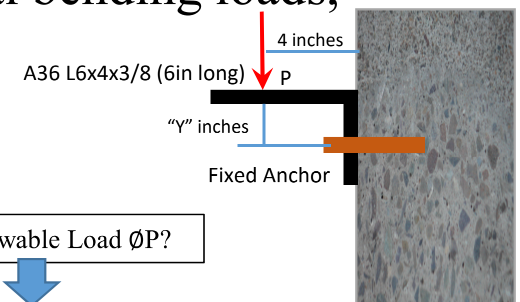
  $A_{nv} = nominal\ area\ subject\ to\ shear = A_{gv} - A_{hv}$

  $A_{nt} = nominal\ area\ subject\ to\ tension$ (remove hole areas) = $A_{gt} - A_{ht}$

  $\emptyset R_n = 0.75R_n$

ANY STEEL COMPONENT– FLEXURE CHECK

AISC Spec. F11 (Rectangular bars & rounds) can be used to evaluate the yield/load capacity of things like: 1) angle legs in local loading, 2) flange local bending loads, 3) seated load yielding, etc.



Consider Boundary Conditions for:

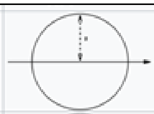
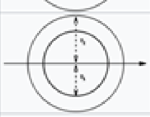
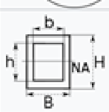
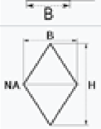
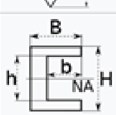
Yielding: $\boldsymbol{\emptyset = 0.9}$

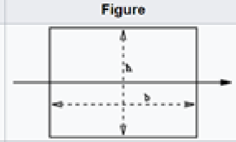
$M_n = M_p = \text{Lesser of} \begin{cases} F_y Z \\ 1.6\, F_y S \end{cases}$

For rectangular sections:

$Z = \frac{bd^2}{4}$ & $S = \frac{bd^2}{6}$

For circles & others: See below

Circle		$S = \frac{\pi r^3}{4} = \frac{\pi d^3}{32}$ [3]
Circular hollow section		$S = \frac{\pi\left(r_2^4 - r_1^4\right)}{4 r_2} = \frac{\pi(d_2^4 - d_1^4)}{32 d_2}$
Rectangular hollow section		$S = \frac{BH^2}{6} - \frac{bh^3}{6H}$
Diamond		$S = \frac{BH^2}{24}$
C-channel		$S = \frac{BH^2}{6} - \frac{bh^3}{6H}$

Description	Figure	Equation
Rectangular section		$Z_P = \frac{bh^2}{4}$ [4][5]
Rectangular hollow section		$Z_P = \frac{bh^2}{4} - (b - 2t)(\frac{h}{2} - t)^2$
For the two flanges of an I-beam with the web excluded[6]		$Z_P = b_1 t_1 y_1 + b_2 t_2 y_2$
For an I Beam including the web		$Z_P = bt_f(d - t_f) + 0.25 t_w (d - 2t_f)^2$
For an I Beam (weak axis)		$Z_P = (b^2 t_f)/2 + 0.25 t_w^2 (d - 2t_f)$
Solid Circle		$Z_P = \frac{d^3}{6}$
Circular hollow section		$Z_P = \frac{d_2^3 - d_1^3}{6}$

Max Allowable Load ØP?

Angle Yielding: Yielding in Long Leg Governs!

$Z_{LongLeg} = \frac{bd^2}{4} = \frac{6(^3/_8)^2}{4} = 0.211$

$S_{LongLeg} = \frac{bd^2}{6} = \frac{6(^3/_8)^2}{6} = 0.141$

$M_n = M_p = \text{Lesser of} \begin{cases} F_y Z = (36)(0.211) = 7.60\ kip - in\ \ (GOVERNS) \\ 1.6\ F_y S = 1.6(36)(0.141) = 8.12\ kip - in \end{cases}$

$\emptyset M_n = 0.9(7.60) = 6.84\ kip - in$

$\emptyset M_n = M_u$ ➔ $6.84\ kip - in = (4in)(P)$ ➔ $P = 1.71\ kips$

Latest version uses (4-(0.5*3/8) Not 4. SO P=1.8 kips

Load on Fixed Anchor:

$n = \frac{Load\ Spacing}{Bolt\ Spacing}$

Applied Shear load per bolt ➔ $b_v = \frac{P_{single\ value}}{n}$

Prying Tension ➔ $T = \frac{P_{single\ value}"X"}{"Y"}$

Prying Tension Per Bolt ➔ $b_a = \frac{T}{n}$

STEEL BEAM – FLEXURE DESIGN

Doubly Symmetric Steel Bent About the **Major** Axis

Calculate Design Factored Loads

Maximum Moment M_u

Calculate I_{req} to Satisfy Deflection Requirements: IBC or ASCE 7-10
*Consider Live Load and Total Load
USE SERVICE LOADS!

Determine C_b Table 3-1
*Consider L_b & Zone 1,2, or 3 bending
Use Table 3-2 or 3-6 Lp and Lr values

<u>Zone 1 Bending</u>
$L_b \le L_p$

Calculate required plastic section modulus $Z_{x,req} = \frac{M_u}{\emptyset_b F_y}$

Use AISC Manual Table 3-2 to select member based on following criterion:
* $Z_{x,req}$, I_{req} , $\emptyset_b M_{px}$

<u>Zone 2 Bending</u>
$L_p < L_b < L_r$

AISC Table 3-6 to get BF (Also consider Table 3-10)
Available flexural strength greater than ultimate factored load and less than factored plastic moment with $C_b = 1.0$
$M_u \le \emptyset_b C_{b1.0} M_n = C_{b1.0}\left(\emptyset_b M_{px} - BF\left(L_b - L_p\right)\right) \le \emptyset_b M_{px}$ [Table 3-6]
Or unconservatively incorporate true C_b
$M_u \le \emptyset_b C_b M_n = \emptyset_b C_b \left(\emptyset_b M_{px} - BF\left(L_b - L_p\right)\right) \le \emptyset_b M_{px}$ [Table 3–6

If initial $C_{b1.0}$ was taken as 1.0, then
Amplify by true C_b
Maximum factored bending moment with C_b = table given value. Table 3-1
$C_b \emptyset_b M_n < \emptyset_b M_{px}$

I_{req} to comply with IBC serviceability deflection limits.
*Specify camber if not compliant

<u>Zone 3 Bending</u>
$L_r < L_b$

Table 3-10 Flex str & Unbraced Length
M_n = Lesser of $F_{cr} S_x$ & $M_p = F_y Z_x$
when

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078\left(\frac{J\,C}{S_x h_0}\right)\left(\frac{L_b}{r_{ts}}\right)^2}$$

AISC Table 3-1
Amplify by true C_b to LTB (not plastic moment)
Maximum factored bending moment with C_b = table given value.
$C_b \emptyset_b M_n < \emptyset_b M_{px}$

STEEL BEAM – FLEXURE DESIGN

I beam and C shape About the **Minor** Axis

Calculate Design Factored Loads

Maximum Moment M_u

Determine M_n to satisfy loading

Compact, Noncompact, Slender? Table B4.1b Case 10 example.

Be careful with "b" dimension and "t"
For I shaped members use half of "b" for flange width

$\frac{b_f}{t_f}$ (or $\frac{b_f}{2t_f}$ for I shaped).

$\lambda = Value \quad \lambda_{pf} = Value \quad \lambda_{rf} = Value$

$Compact: \quad \lambda < \lambda_{pf}$

$Noncompact: \lambda_{pf} < \lambda < \lambda_{rf}$

$Slender: \quad \lambda_{rf} < \lambda$

Compact Flanges:
Flexural Yield Strength Controls

$M_n = M_p$ = Lesser of $F_y Z_y$

$1.6\ F_y S_y$

Also see Table 3-4 $\emptyset_b M_{py}$

$øM_n > M_u$?

Noncompact Flanges:
Check Flexural Yield & Flange Local Buckling

M_p = Lesser of $F_y Z_y$

$1.6\ F_y S_y$

$M_n = M_p - (M_p - 0.7F_y S_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$

Also see Table 3-4 $\emptyset_b M_{py}$

$øM_n > M_u$?

Slender Flanges:
Check Critical Load

$M_n = F_{cr} S_y$

$F_{cr} = \frac{0.69\ E}{\left(\frac{b}{2t_f}\right)^2}$

AISC F6 pg 16.1 – 56

b = for I-shaped members, use half of the full-flange width.

$øM_n > M_u$?

Shortcut: AISC Table 3-4

STEEL BEAM – FLEXURE DESIGN

Square and Rectangular HSS and Box Members

Calculate Design Factored Loads

Maximum Moment M_u

Determine M_n to satisfy loading

<u>Identify Governing Limit State for Moment Capacity</u>

Compact, Noncompact, or Slender Flanges? Table B4.1b Case 17.

$\lambda = Value$ * b/t listed in the AISC Chapter 1 tables.

$\lambda_{pf} = Value$ $\lambda_{rf} = Value$

$Compact$: $\lambda < \lambda_{pf}$

$Noncompact$: $\lambda_{pf} < \lambda < \lambda_{rf}$

$Slender$: $\lambda_{rf} < \lambda$

Compact or Noncompact Web? Table B4.1b Case 19

$Compact$: $\lambda < \lambda_{pf}$

$Noncompact$: $\lambda_{pf} < \lambda < \lambda_{rf}$

CONTROLLING LIMIT STATES: Identify Applicable Flange and Web Conditions

Table 1-12A for Simplified Compactness Criterion

<u>Flange Conditions</u>

<u>A) Compact Flanges:</u>
Flexural Yield Strength Controls

$M_n = M_p = F_y Z$

Also see Table 3-12 $\emptyset_b M_n$

$\text{ø}M_n > M_u$?

<u>B) Noncompact Flanges:</u>
Check Flange Local Buckling

$M_p = F_y Z_{Applicable}$

$$M_n = M_p - (M_p - F_y S)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \leq M_p$$

Also see Table 3-12 $\emptyset_b M_n$

$\text{ø}M_n > M_u$?

<u>C) Slender Flanges:</u>

$M_n = F_y S_e$

When S_e is the effective section modulus evaluated w/:

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left(1 - \frac{0.38}{\frac{b}{t_f}}\sqrt{\frac{E}{F_y}}\right) \leq \text{b} \quad \text{(AISC Eq. F7-4)}$$

See AISC 14th Ed. design example F.8B

Compare to Table 3-12 or 3-13

$\text{ø}M_n > M_u$?

<u>Web Conditions</u>

<u>D) Compact Web:</u>
Flexural Yield Strength Controls

$M_n = M_p = F_y Z_x$

Also see Table 3-12 $\emptyset_b M_n$

$\text{ø}M_n > M_u$?

<u>E) Noncompact Web:</u>
Check Web Local Buckling

$$M_n = M_p - (M_p - F_y S)\left(0.305\left(\frac{h}{t_w}\right)\sqrt{\frac{F_y}{E}} - 0.738\right) \leq M_p$$

Also see Table 3-12 $\emptyset_b M_n$

STEEL BEAM – FLEXURE DESIGN

Round HSS and Pipe Members

Calculate Design Factored Loads

↓

Maximum Moment M_u

↓

Determine M_n to satisfy loading

↓

Identify Governing Limit State for Moment Capacity

Compact, Noncompact, or Slender Round HSS Section? Table B4.1b Case 20.

$\lambda = Value$ * D/t listed in the AISC Chapter 1 tables.

$\lambda_p = Value$ $\lambda_r = Value$

$Compact$: $\lambda < \lambda_p$

$Noncompact$: $\lambda_p < \lambda < \lambda_r$

$Slender$: $\lambda_r < \lambda$

HSS Conditions

A) Compact HSS:
Flexural Yield Strength Controls

$M_n = M_p = F_y Z$

Also see Table 3-14 $\emptyset_b M_n$

↓

$øM_n > M_u$?

B) Noncompact HSS:
Check Local Buckling

$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y\right) S$

Also see Table 3-14 $\emptyset_b M_n$

↓

$øM_n > M_u$?

C) Slender HSS:
Check Critical Flexural Stress

$$M_n = F_{cr} S$$

$$F_{cr} = \frac{0.33E}{\frac{D}{t}}$$

Also see Table 3-14 $\emptyset_b M_n$

STEEL BEAM – FLEXURE DESIGN

Tees and Double Angle Members Loaded in Plane of Symmetry

Determine Governing M_n. Lowest of A through D:

Calculate Design Factored Loads

↓

Maximum Moment M_u

↓

Determine M_n to satisfy loading

↓

Compact, Noncompact, or Slender Member? Table B4.1b Cases 10 & 14.

$\lambda = Value$ * b/t & d/t listed in the AISC Chapter 1 tables. $\frac{b_f}{2t_f}$

$\lambda_p = Value$ $\lambda_r = Value$

$Compact$: $\lambda < \lambda_p$

$Noncompact$: $\lambda_p < \lambda < \lambda_r$

$Slender$: $\lambda_r < \lambda$

A) Flexural Yield Strength:

$M_n = M_p =$ stem in tension: min of: $F_y Z_x$, $1.6 F_y S_x$

stem in comp.: min of: $F_y Z_x$, $F_y S_x$

↓

$øM_n > M_u$?

B) Lateral Torsional Buckling:

$$M_n = M_{cr} = \left(\frac{\pi\sqrt{EI_yGJ}}{L_b}\right)\left(B + \sqrt{1 + B^2}\right)$$

Where: $B = \pm 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}}$ "+" for stem in tension

"-" for stem in comp.

↓

$øM_n > M_u$?

C) Flange Local Comp. Buckling:

Compact Flange: N/A

Noncompact Flange:

$$M_n = M_p - \left(M_p - 0.7F_yS_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \leq 1.6F_yS_x$$

$Where:$ $S_{xc} = \frac{I_{x-x}}{\bar{y}_{x-x}}$ & $M_p = F_X Z_X$ for comp. yield

Slender Flange:

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f}\right)^2}$$

↓

$øM_n > M_u$?

D) Stem Local Comp. Buckling:

$M_n = F_{cr} S_x$

If: $\frac{d}{t_w} \leq 0.84\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = F_y$

$0.84\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 1.03\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = \left(2.55 - 1.84\left(\frac{d}{t_w}\right)\sqrt{\frac{F_y}{E}}\right)F_y$

$1.03\sqrt{\frac{E}{F_y}} \leq \frac{d}{t_w}$ ➔ $F_{cr} = \frac{0.69E}{\left(\frac{d}{t_w}\right)^2}$

↓

$øM_n > M_u$?

STEEL BEAM – SHEAR DESIGN: AISC Ch. G

W, S, & HP Shapes up to F_y of 50 ksi:

Calculate Design Factored Loads

Maximum Shear V_u

Determine V_n to satisfy loading

$\phi_v V_n = \phi_v 0.6 F_y A_w C_v$
Usually $A_w = d\, t_w$

For members with
$\frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}}$ → $\phi_v = 1.00$ $C_v = 1.0$
This includes all 50+ ksi W,S,&HP shapes except: W44x230, W40x149, W36x135, W33x118, W30x90, W24x55, W16x26, & W12x14

Otherwise: $\phi_v = 0.90$ $C_v = Calculated$ by AISC Eq. G2-3, G2-4, & G2-5.

Rectangle HSS & Box Shapes:

Determine V_n to satisfy loading

$\phi_v V_n = \phi_v 0.6 F_y A_w C_v$
Usually $A_w = 2h\, t_w$
$h = d - 3t_w$
$t_w = t$
$k_v = 5$
$\phi_v = 0.9$
C_v calculated by AISC Eq. G2-3, G2-4, & G2-5.

Round HSS

Determine V_n to satisfy loading

$$\phi_v V_n = \frac{\phi_v F_{cr} A_g}{2}$$

$F_{cr} = largest\ of$ $\dfrac{1.6E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{5/4}}$

$\dfrac{0.78E}{\left(\frac{D}{t}\right)^{3/2}}$

But less than $0.6F_y$

Usually $L_v = distance\ from\ maximum\ to\ zero\ shear\ force\ along\ beam$

$\phi_v = 0.9$

AISC Eq. G2-3, G2-4, & G2-5.

STEEL Compression/Column Design

Nominal Compressive Strength

- $P_n = F_{cr}A_g$ [AISC E3-1]
- For LRFD: $\emptyset_c P_n = \emptyset_c F_{cr} A_g$
- K = effective length factor [AISC Apdx. 7]

Calculating Effective Length [**KL**] for each Axis (XX, YY, ZZ)

- Largest KL/r governs (keep under 200)
- If member sizes are known, calculate G_A & G_B
- Apply them to the proper Alignment Chart (Sway or Nonsway) to get "K"

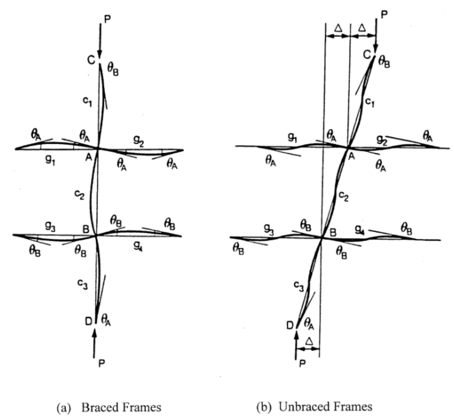
$$G = \frac{\sum \frac{EI_{col}}{L_{col}}}{\sum \frac{EI_{girder}}{L_{girder}}} = \frac{\sum \frac{I_{col}}{L_{col}}}{\sum \frac{I_{girder}}{L_{girder}}} \text{ (if all steel is same material)}$$

Slender Elements ≠ Slender Members

Check Table B4.1a for slender elements (Example: λ_{rw})

Approximate Values of Effective Length Factor, K

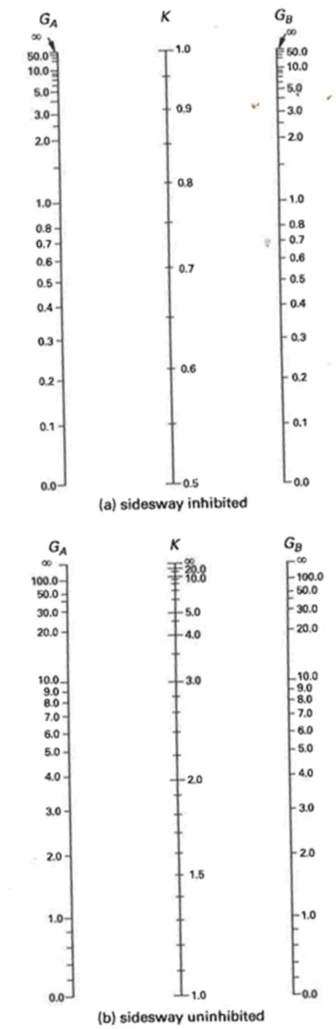
End 1	End 2	Theoretical K	Recommended Design K-Value
Built-in (rotation fixed, translation fixed)	Built-in	0.50	0.65
Built-in	Pinned (rotation free, translation fixed)	0.70	0.80
Built-in	Rotation fixed, translation free	1.00	1.20
Built-in	free	2.00	2.10
Pinned	Pinned	1.00	1.00
Pinned	Rotation fixed, translation free	2.00	2.00



IGURE 17.3: Subassemblage models for K-factors of framed columns.

$$G_B = \frac{\sum_B (E_c I_c / L_c)}{\sum_B (E_g I_g / L_g)}$$

Alignment Charts for Determining Effective Length Factor, K



STEEL Compression/Column Design – AISC Spec E3

Compressive Strength for Flexural Members w/out Slender Elements

Slender elements can buckle before overall member buckling.

Nominal Compressive Strength

- $P_n = F_{cr} A_g$ [AISC E3-1]
- For LRFD: $\emptyset_c P_n = \emptyset_c F_{cr} A_g$
- $\emptyset_c$ = 0.9
- AISC Table 4-22 Design Str. For Various KL/r Values

Calculating Effective Length [**KL**] for each Axis (XX, YY, ZZ)

- Largest KL/r governs (keep under 200)
- If member sizes are known, calculate G_A & G_B
- Apply them to the proper Alignment Chart (Sway or Nonsway) to get "K"

When Effective slenderness ratio

$$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}} \quad \text{or} \quad \frac{F_y}{F_e} \leq 2.25$$

$$F_{cr} = 0.658^{\left(\frac{F_y}{F_e}\right)} F_y$$

When Effective slenderness ratio

$$\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}} \quad \text{or} \quad \frac{F_y}{F_e} > 2.25$$

$$F_{cr} = 0.877 F_e$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$\emptyset_c P_n = \emptyset_c F_{cr} A_g \geq P_u?$$

STEEL Compression/Column Design – AISC Spec E3

"Select the Lightest I shaped Column" Style Design (without slender elements)

Determine P_u
- Factored Loadings

Determine Effective Length KL
- Consider each axis x-x, y-y, & z-z
- Longest governs

Determine Effective Length KL
- Consider each axis x-x, y-y, & z-z
- Longest governs

AISC Table 4-1 to select member
- $\emptyset_c P_n$
- Verify $K_x L_x{}' = \frac{K_x L_x}{\frac{r_x}{r_y}} < K_y L_y$
  - Otherwise: Verify column @ length $K_x L_x{}'$
  - If it doesn't verify, select a larger column size

$\emptyset_c P_n = \emptyset_c F_{cr} A_g \geq P_u$?

"Determine the Design Load Capacity" Style Design (without slender elements)

Determine the Slenderness Ratio
- Largest KL/r governs
- So check $\frac{K_x L_x}{r_x}$ & $\frac{K_y L_y}{r_y}$

Member in Elastic or Inelastic Range for F_{cr}?

When Effective slenderness ratio $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} \leq 2.25$

$F_{cr} = 0.658^{\left(\frac{F_y}{F_e}\right)} F_y$

When Effective slenderness ratio $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} > 2.25$

$F_{cr} = 0.877 F_e$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

$\emptyset_c P_n = \emptyset_c F_{cr} A_g$

Alternate Procedure!

Determine the Slenderness Ratio
- Largest KL/r governs
- So check $\frac{K_x L_x}{r_x}$ & $\frac{K_y L_y}{r_y}$

AISC Table 4-22
- Available $\emptyset_c F_{cr}$ by KL/r

$\emptyset_c P_n = \emptyset_c F_{cr} A_g$

STEEL Compression/Column Design – AISC Spec E4

Torsional or Flex/Torsional Buckling of Members w/out Slender Elements

Strength Governed by Nominal Comp. Str.
- $P_n = F_{cr} A_g$

F_{cr} is categorized as
- Double angle and T-shaped comp. members
- All other cases
  - Doubly symmetric members
  - Singly symmetric members
  - Unsymmetrical members

Double Angle & T-Shaped Comp. Members

Lowest value of F_{cr} governs:
- Standard Flexural Buckling in X & Y (previous slide)
- Torsional and Flexural/Torsional Buckle

Torsional & Flex/Torsion Buckle
See AISC Table 4-7 thru 4-12 for shortcut

$$F_{cr} = \left(\frac{F_{cr,y} + F_{cr,z}}{2H}\right)\left(1 - \sqrt{1 - \frac{4F_{cr,y}F_{cr,z}H}{\left(F_{cr,y} + F_{cr,z}\right)^2}}\right)$$

Where:

$F_{cr,y}$ = traditional F_{cr} from AISC Eq. E3-2 & E3-3

$F_{cr,z} = \frac{GJ}{A_g \bar{r}_0^2}$

$\bar{r}_0^2$ = polar radius of gyration about shear center

$= X_0^2 + Y_0^2 + \frac{I_x + I_y}{A_g}$ where: X_0 & Y_0 are distances between shear center and centroid.

Shear center for double angle and T shaped is at web-flange intersection. This makes it $Y_0 = \bar{Y} - {}^{t_f}/_2$

$H = 1 - \frac{X_0^2 + Y_0^2}{\bar{r}_0^2}$ ($X_0^2 = 0$ for members symmetrical about Y-axis)

($Y_0^2 = 0$ for members symmetrical about X-axis)

Other Cases

See AISC Table 4-7 thru 4-12 for shortcut

Lowest value of F_{cr} governs:
- F_e for doubly symmetric members

$$F_e = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\left(\frac{1}{I_x + I_y}\right)$$

- F_e for doubly symmetric members

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left(1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right)$$

Where: $F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2}$

$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2}$

$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\left(\frac{1}{A_g \bar{r}_0^2}\right)$

When Effective slenderness ratio $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$

$F_{cr} = 0.658^{\left(\frac{F_y}{F_e}\right)} F_y$

When Effective slenderness ratio $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$

$F_{cr} = 0.877 F_e$

$$\emptyset_c P_n = \emptyset_c F_{cr} A_g \geq P_u?$$

STEEL Compression/Column Design – AISC Spec E4

Torsional or Flex/Torsional Buckling of Members with Slender Elements

Strength Governed by Nominal Comp. Str.

- $P_n = F_{cr} A_g$
- Just use Table 4 - #

STEEL Compression/Column Design – AISC Spec E4

Columns w/ Cover Plates Bolted or Welded w/ equivalent "r" (pg 178 Mccormacka Textbook)

STEEL Compression/Column Design – AISC Spec E

Design of Single Angles in Comp. Use CHAPTER "H" IF These Conditions Not Satisfied

Eccentricity may be neglected when:

- Members are loaded at the ends in compression through the same leg
- Members are attached by welding or by at least 2 bolts per connection
- There are not intermediate transverse loads

Choose proper effective member slenderness ratios:

Case 1 – Individual members & web members of planar trusses w/ adjacent web members attached to the same side of the gusset plate or chord

1A) **For equal leg angles and unequal leg angles connected through longer leg:**

- Calculate L/r_x

if $0 \leq L/r_x \leq 80$: $\frac{KL}{r} = 72 + 0.75\left(\frac{L}{r_x}\right)$

if $L/r_x > 80$: $\frac{KL}{r} = 32 + 1.25\left(\frac{L}{r_x}\right) \leq 200$

1B) For unequal leg angles with ratio of long leg/short leg length less than 1.7 and connected through shorter leg:

- Calculate L/r_x

if $0 \leq L/r_x \leq 80$: $\frac{KL}{r} = 72 + 0.75\left(\frac{L}{r_x}\right) + 4\left(\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right) \leq 0.95\left(\frac{L}{r_z}\right)$

if $L/r_x > 80$: $\frac{KL}{r} = 32 + 1.25\left(\frac{L}{r_x}\right) + 4\left(\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right) \leq 0.95\left(\frac{L}{r_z}\right)$

Choose proper effective member slenderness ratios:

Case 2 – Web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord

2A) For equal leg angles and unequal leg angles connected through longer leg:

- Calculate L/r_x

if $0 \leq L/r_x \leq 75$: $\frac{KL}{r} = 60 + 0.80\left(\frac{L}{r_x}\right)$

if $L/r_x > 75$: $\frac{KL}{r} = 45 + 1.00\left(\frac{L}{r_x}\right) \leq 200$

2B) For unequal leg angles with ratio of long leg/short leg length less than 1.7 and connected through shorter leg:

- Calculate L/r_x

if $0 \leq L/r_x \leq 75$: $\frac{KL}{r} = 60 + 0.80\left(\frac{L}{r_x}\right) + 6\left(\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right) \leq 0.82\left(\frac{L}{r_z}\right)$

if $L/r_x > 75$: $\frac{KL}{r} = 45 + 1.00\left(\frac{L}{r_x}\right) + 6\left(\left(\frac{b_{long}}{b_{short}}\right)^2 - 1\right) \leq 0.82\left(\frac{L}{r_z}\right)$

SHORTCUT!

After getting $\frac{KL}{r}$ multiply by member r_z so

$KL = \left(\frac{KL}{r}\right) r_z$

Use Table 4-11 to interpolate $\emptyset_c P_n$

or

When Effective slenderness ratio $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} \leq 2.25$

$$F_{cr} = 0.658^{\left(\frac{F_y}{F_e}\right)} F_y$$

When Effective slenderness ratio $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} > 2.25$

$$F_{cr} = 0.877 F_e$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$\emptyset_c P_n = \emptyset_c F_{cr} A_g$$

BEST SHORTCUT!

After getting $\frac{KL}{r}$

Use Table 4-22 to interpolate $\emptyset_c P_n$

Base Plate and Anchor Bolt Design

Base Plate and Anchor Bolt Design

AISC Part 14 ➔ Bearing Plates & Anchor Rods
- Table 10-1 ➔ Plate finishing allowances
- Table 10-2 ➔ Max. Anchor-Rod holes in Base Plates & WASHER THCK

AISC Design Guide 1
- Washer do not need to be hardened
- Anchor-rod sizing and layout
  - Use ¾" diameter ASTM F1554 Gr. 36 whenever possible.
- Tolerances in ACI 117 & AISC 303 Section 7.5.1

Column Setting Methods
- Setting Nut and Washer Method
  - Typical w/ 4-rod layout
  - Be sure to check push-out at bottom of footing
- Setting Plate Method (AISC M4.4)
  - Use a setting plate to establish elevations
  - Can be repeated if errors occur
- Shim Stack Method
  - Use shims to stabilize each size of base plate

Grout
- Should have a compressive strength at least twice that of the concrete foundation

OSHA
- Requires 4-rod minimum per base-plate connection.
- Excluding columns/post less than 300 lbs.

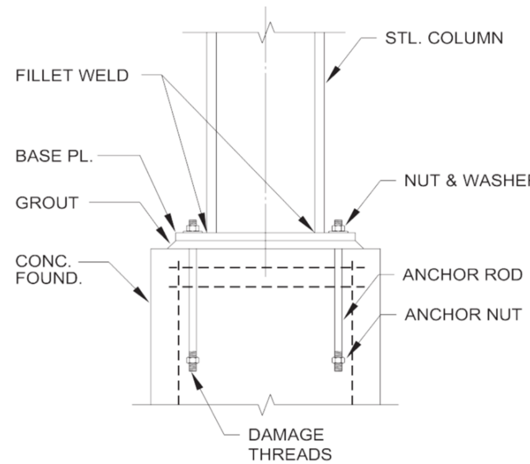


Table 2.3. Recommended Sizes for Anchor Rod Holes in Base Plates

Anchor Rod Diameter, in.	Hole Diameter, in.	Min. Washer Dimension, in.	Min. Washer Thickness, in.
¾	1 5/16	2	¼
⅞	1 9/16	2½	5/16
1	1 13/16	3	⅜
1¼	2 1/16	3	½
1½	2 5/16	3½	½
1¾	2¾	4	⅝
2	3¼	5	¾
2½	3¾	5½	⅞

Notes: 1. Circular or square washers meeting the size shown are acceptable.
2. Adequate clearance must be provided for the washer size selected.
3. See discussion in Section 2.6 regarding the use of alternate 1 1/16-in. hole size for ¾-in.-diameter anchor rods, with plates less than 1¼-in. thick.

Base Plate and Anchor Bolt Design

Base Plate and Anchor Bolt Design

AISC Design Guide 1

- Five considerations of AISC DG1
  - Concentric Compressive Axial Loads
  - Tensile Axial Loads
  - Column Base Plates w/ Small Moments
  - Column Base Plates w/ Large Moments
  - Shear Design

Concentric Compressive Axial Loads

- ACI 318 10.14 ➔ Design Bearing Strength = $\emptyset(0.85 f_c{}' A_1)$ $\emptyset = 0.65$
  - May multiply by $\sqrt{A_1/A_2} \leq 2$ if support wider than base plate on all sides
- AISC J8 ➔Design Bearing Strength = $P_p = \emptyset(0.85 f_c{}' A_1)$ when using whole support
  - $P_p = \emptyset(0.85 f_c{}' A_1)\sqrt{A_1/A_2} \leq 1.7 f_c{}' A_1$ $\emptyset = 0.60$
- Required bearing stress under base plate AISC DG1 (pg. 16-18)
  - Case 1: $A_1 = A_2$
    - $A_{1req} = \frac{P_u}{\emptyset_c 0.85 f_c{}'}$
  - Case 2: $A_2 \geq 4A_1$
    - $A_{1req} = \frac{P_u}{2\emptyset_c 0.85 f_c{}'}$
  - Case 3: $A_1 < A_2 < 4\,A_1$
    - $A_{1req} = \frac{P_u}{2\emptyset_c 0.85 f_c{}'}$

AISC Design Guide 1

- See Design Examples for the Five considerations of AISC DG1
  - Concentric Compressive Axial Loads
  - Tensile Axial Loads
  - Column Base Plates w/ Small Moments
  - Column Base Plates w/ Large Moments
  - Shear Design

STEEL Column Splice Design

AISC Ch14
Problem 7 from Structural Engr Solved Problems
Cives Steel Books from Don

Combined Stress Member Design – AISC Spec H

Combined Stress Member Design covers:
- H1 – Doubly & Singly Symmetrical Members Subject to Flexure and Axial Force
- H2 – Unsymmetrical & Other Members Subject to Flexure and Axial Force
- H3 – Members Under Torsion & Combined Tors, Flex, Shear, & or Axial Force
- H4 – Rupture of Flanges with Holes Subject to Tension

H1 – Doubly & Singly Symmetrical Members Subject to Flexure and Axial Force

Doubly & Singly Symmetric: Design for **Axial Force** and Flexure or **Biaxial Flexure**
The following general set of equations [AISC H1-1a & 1b] govern (ø and Ω applied prior to inserting in the equations).

For $\frac{P_r}{P_c} \geq 0.2$ $\quad \frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$ [H1-1a]

For $\frac{P_r}{P_c} < 0.2$ $\quad \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$ [H1-1b]

Where:
- $P_r = required\ axial\ compressive\ strength$
- $P_c = available\ axial\ compressive\ strength$
- $M_r = required\ flexural\ strength$
- M_c = available flexural strength

Doubly Symmetric: Design for **Tension** and Flexure or **Biaxial Flexure**
Use Equations H1-1a & H1-1b, with the caveat

C_b from Ch. F may be multiplied by $\sqrt{1 + \frac{\alpha P_r}{P_{ey}}}$ where: $P_{ey} = \frac{\pi^2 E I_y}{L_b^2}$;

$\alpha = 1.00\ LRFD = 1.60\ ASD$

Despite this: You can use a conservative C_b of 1.0

Doubly Symmetric & Singly Symmetric Constrained to bend about a geometric Axis:
Design for **Tension** and Flexure or **Biaxial Flexure**
Use Equations H1-1a & H1-1b, with the caveat

C_b from Ch. F may be multiplied by $\sqrt{1 + \frac{\alpha P_r}{P_{ey}}}$ where: $P_{ey} = \frac{\pi^2 E I_y}{L_b^2}$; $\alpha = 1.00\ LRFD = 1.60\ ASD$

Despite this: You can use a conservative C_b of 1.0

Combined Stress Member Design – AISC Spec H

Beam-Columns Using Second Order Analysis

- Pg 350 of Mccormack Textbook
- Magnification factors
- Direct analysis method
- Effective length method
- Approximate 2^{nd} order methods

Bolted Connection Design – AISC Spec J

Bolted Connections – AISC Chapter J (K for HSS)

- AISC Part 7 – Design Considerations for Bolts
- AISC Part 8 – Design Considerations for Welds
- AISC Part 9 – Design of Connecting Elements
- AISC Part 10 – Design of Simple Shear Connections
- AISC Part 11 – Design of Partially Restrained Moment Connections
- AISC Part 12 – Design of Fully Restrained (FR) Moment Connections
- AISC Part 13 – Design of Bracing Connections and Truss Connections

AISC Tables:

Table 7-1 – Available Shear Str. Of Bolts: Single or Double Shear Planes
Table 7-2 – Available Tensile Strength of Bolts
Table 7-3 – Slip Critical Conn: Shear Str., if slip service limit-state
Table 7-4 – Available Bearing Str. @ Bolt Holes: Bearing for Support/Supported Elements, based on Bolt Spacing. Section J3.10
Table 7-5 – Available Bearing Str. @ Bolt Holes: Bearing for Support/Supported Elements, based on Edge Distance. Section J3.10
Table J3.1 – Minimum Bolt Pretension
Table J3.2 – Nominal Str. of Fasteners & Threaded Parts
Table J3.3 – Nominal Hole Dimensions
Table J3.4 – Minimum Edge Distance from Center of Standard Hole to Edge of Connected Part
Table J3.5 – Values of Edge Distance Increment C2

Important Sections

AISC Spec J3.1 – Group A (A325) & Group B (A490) high strength bolts
AISC Spec J3.3 – Minimum Bolt Spacing (not less than $2\,^{2}/_{3}\,d_b$, $3d_b$ preferred)
AISC Spec J3.4 – Minimum Edge Spacing (Table 3.4 per Section J3.10)
AISC Spec J3.5 – Max Edge Distance (bolt center to edge) = $12(t_{conn.part}) \le 6"$
– Max Bolt Long. Spacing = unpainted/painted corrosion resist ➔ $24d_b \le 12"$
Weathering Steel ➔ $14d_b \le 7"$

Bolt Bearing Capacity $\emptyset_v = 0.75$

Check each piece (although it may be obvious that one governs based on thickness)!

Shortcut: AISC Table 7-4 or 7-5 ➔ $\emptyset_v r_n = \frac{bolt\ kips}{1\ inch\ of\ thickness}$

$\emptyset_v R_n = n[(\emptyset_v r_n)(thickness_{Thinnest mtrl})]$ n = # of bolts in shear

THE LONG WAY IS REQUIRED IF BOLT SPACINGS DON'T CONFORM TO AISC TABLES!!

Long Way: when bolt spacing doesn't conform to $2\frac{2}{3}d_b$ or 3" (AISC Table 7-4 measured from center to center) or edge distance doesn't meet $1\frac{1}{4}$" or 2" (AISC Table 7-5 measured center to edge).

AISC J3.10 Specifications: $\emptyset_v = 0.75$

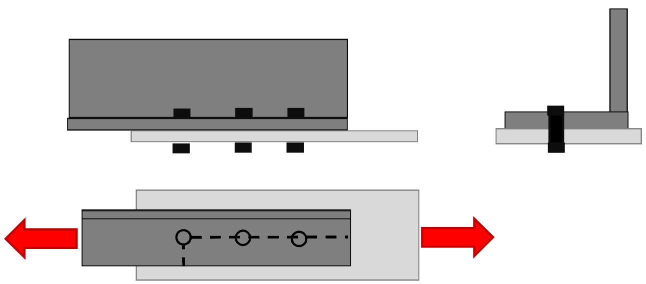
- For a bolted connection w/ standard, oversize, or short-slotted holes, independent of load direction, or a long-slotted hole w/ slot parallel to direction of bearing force **per bolt**:
  - When service load deformation @ hole is considered (δ< 0.25")
    $R_n = 1.2 l_c t F_u \le 2.4 d t F_u$ (shear tearout vs ovalization)
  - When deformation is not a design consideration
    $R_n = 1.5 l_c t F_u \le 3.0 d t F_u$
- For a bolt in a connection w/ long-slotted holes w/ slot perpendicular to direction of force:
  $R_n = 1.0 l_c t F_u \le 2.0 d t F_u$

l_c = clear distance in direction of force between edge of hole and edge of adjacent hole $(S - d_{hole})$ or edge of mtrl $(Ledge\ Disance - 0.5d_{hole})$
$d_{hole} = d_{bolt} + 0.125"$
d = nominal bolt diameter
t = thickness of connecting material bearing against bolt.

- Total bearing resistance shall be taken as the sum of the bearing resistance of individual bolts
  - $\sum \emptyset R_n$
- For HSS connections w/ bolts that pass through, see AISC J

Bolted Connection Design – AISC Spec J

Simple Tension Bolted Lap Splice



<u>Bolted Connections – Tension Considerations</u>
- Gross Section Yield of Each Piece
- Net Section Rupture of Each Piece
- Bolts in Single/Double Shear
- Bolts in Bearing
- Block Shear Rupture

<u>Identify Preliminary Design Parameters</u>
thicknesses, bolt sizes, Fy, Fu, dims.

<u>Gross Section Yield</u> Check each piece!
$\emptyset_t T_n = \emptyset_t A_g F_y$ $\emptyset_t$=0.9

<u>Net Section Rupture</u> Check each piece!
$\emptyset_t T_n = \emptyset_t A_e F_u$ $\emptyset_t$=0.75
Be sure to consider shear lag. $A_e = UA_n$
In the case of this plate, U=1.0 for full contact. However, use D3.1 for the angle (case 2 (best) or 8).

<u>Check Bolts in Shear</u>
AISC Table 7-1 ➔ $\emptyset_v r_n$ in kips/bolt
$\emptyset_v R_n = n\emptyset_v r_n$ n = # of bolts in shear

<u>Bolt Bearing Capacity</u> (Shortcut Used: See previous Slide for Methods)
Use long way if shortcut won't work for spacings
Check each piece (although it may be obvious that one governs based on thickness)!
AISC Table 7-4 or 7-5 ➔ $\emptyset_v r_n = \frac{bolt\ kips}{1\ inch\ of\ thickness}$
$\emptyset_v R_n = n[(\emptyset_v r_n)(thickness_{Thinnestmtrl})]$ n = # of bolts in shear

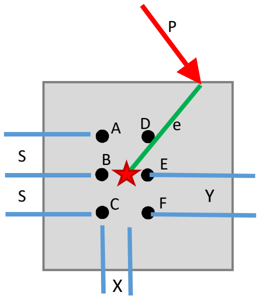
<u>Block Shear:</u>
$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$ [AISC EQ J4-5]
$U_{bs} = 1$ when tension stress is uniform, 0.85 otherwise
$A_{nv} = nominal\ area\ subject\ to\ shear = A_{gv} - A_{hv}$
$A_{nt} = nominal\ area\ subject\ to\ tension$
$\emptyset R_n = 0.75R_n$

Failure Mode	LRFD (Kips)
Gross Section Yield (piece 1)	#
Net Section Rupture (piece 1)	#
Gross Section Yield (piece 2)	#
Net Section Rupture (piece 2)	#
Bolts in Shear	#
Bolts in Bearing (piece 1 or 2)	#
Block Shear Rupture	#

<u>Lowest Value GOVERNS</u>

Bolted Connection Design – AISC Spec J

Bracket Connection w/ Eccentric Shear (no Tension)



Bolted Connections – Considerations
- Instantaneous Center of Rotation Method
- Elastic Method

Instantaneous Center of Rotation Method
- ICR is more accurate but more difficult w/out AISC Tables
- Combined shear forces from axial load and rotational moment are = to force produced by rotation alone about some point. This point is the IC of R, and its location depends on where and what direction the axial load is applied and bolts are arranged.
- Resistance of each fastener acts in the direction perpendicular to a line from the center of the fastener to the instantaneous center.
- Coefficients are in Table 7-7 to 7-14

Elastic Method
- Sometimes called Vector Analysis Method
- More conservative results (sometimes overly conservative)
- It assumes
  - Equal share of vertical component for each bolt
  - Equal share of horizontal component for each bolt
  - Proportional share of eccentric moment portion of load (depends of bolt distance from centroid of bolt group).

ICR Method

Calculate Design Loads
$P_u = 1.2D + 1.6L$

Calculate Eccentricity e_x
Distance from center of bolt group to load point.

AISC Table 7-6 to 7-13
Get "C" for bolt group based on "s", "e_x", "load angle", "n = # of bolts in a given row". Interpolate if necessary
e_x is horizontal distance from Bolt CG!!!
Diagonal Loads need to be projected to where they intersect the horizontal line.

AISC Table 7-1
Available Shear Str. of 1 bolt = $\emptyset_v r_n$

Verify C_{min}
C_{min} given in AISC Table 7-6 to 7-13.
$C_{min} = \frac{P_u}{\emptyset_v r_n}$ ➔ $C_{min} \leq$ "C" from table = good

Elastic Method

Calculate Design Loads
$P_u = 1.2D + 1.6L$

Calculate Moment Created by Eccentricity
$M_e = P_u e$ (e = perp. dist. Of CG to load)

Calculate $\sum d^2$ (Sum dist. of all bolts to CG of bolt group)
$\sum d^2 = \sum X^2 + \sum Y^2$
Note: X & Y distances depend on CG.

Calc. Hor. & Vert. shear on each bolt (from P components)
$R_{vy} = \frac{P_{uy}}{n}$ kips/bolt $R_{vx} = \frac{P_{ux}}{n}$ kips/bolt (n=6 above)

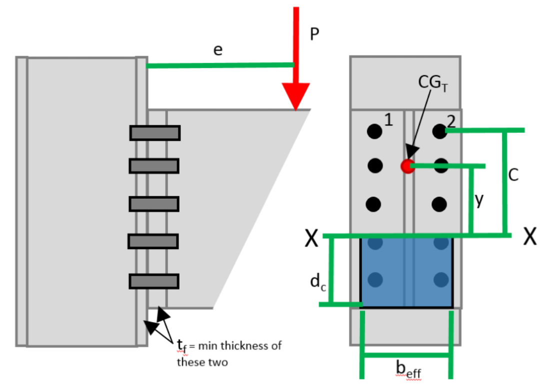
Calculate Hor. & Vert. shear on each bolt (from M_{ex})
$R_x = \frac{M\,y}{\sum d^2}$ kips/bolt $R_y = \frac{M\,x}{\sum d^2}$

Governing Bolt w/ Max. Resultant Shear Force "R"
Isolate bolt with max. shear value "R"
$R = \sqrt{(R_x + R_{vx})^2 + (R_y + R_{vy})^2}$

Shear Check
R ≤ $\emptyset_v r_n$ from AISC Table 7-1 ➔ Good.

Bolted Connection Design – AISC Spec J

Combined Shear & Tension in Bearing Connections: Bracket Case I



Combined Shear & Tension

- AISC Sec. J3.7 – Combined V&T in Bearing Connections
- AISC Sec. J3.9 – Combined V&T in Slip Crit. Connections

When the required stress in either shear or tension is less the or equal to 30% of the available stress, the effects of combined stress need not be investigated.

Available tensile strength of bolts w/ tension & shear (R_n)

$R_n = F'_{nt} A_b$ $\quad\quad$ $\emptyset R_n = 0.75 R_n$ for LRFD

Where: $F'_{nt} = 1.3F_{nt} - \left(\frac{F_{nt}}{\emptyset F_{nv}}\right) f_{rv} \leq F_{nt}$

$F_{nt} = nominal\ tensile\ stress\ from\ AISC\ Table\ J3.2$

$F_{nv} = nominal\ shear\ stress\ from\ AISC\ Table\ J3.2$

$f_{rv} = required\ shear\ stress\ using\ LRFD$

A_b = nominal unthreaded body area of bolt or threaded part

Method of Design

AISC Ch 7
Case I (N.A. not @ C.G) or II (N.A. @ C.G.)

AISC Ch. 7 Case I

Shear force Per Bolt:

$r_{uv} = \frac{P_u}{n}$

Tension per Bolt:

1) Estimate NA @ d_c= depth/6
2) Establish horizontal size of compression block

$b_{eff} = 8t_f \leq b_f$

3) Verify by taking moment about proposed NA

$(\sum A_b)y = b_{eff} d_c \left(\frac{d_c}{2}\right)$ $\quad$ *Adjust NA accordingly*

4) Establish I_x

$I_x = A_b\left(\sum d_y\right)^2 + \frac{b_{eff}(d_c)^3}{3}$ $\quad$ Where: d_y= bolt distance to NA

5) Tensile force in farthest bolt

$r_{ut} = \left(\frac{P_u\, e\, c}{I_x}\right) A_b$

Shear & Tensile Stress in Bolts at 30% Threshold?:

$f_{rv} = \frac{r_{uv}}{A_b}$ $\quad\quad$ $f_t = \frac{r_{ut}}{A_b}$

Is $f_{rv} \geq 0.3\ \emptyset F_{nv}$? $\quad\quad$ $\emptyset = 0.75\ for\ both\ cases$

Is $f_t \geq 0.3\ \emptyset F_{nt}$?

If either exceed 30% ➔ Consider combined V&T.

If not ➔ Done.

Available Tensile Strength (Revisited for Combined V&T):

$$F'_{nt} = 1.3F_{nt} - \left(\frac{F_{nt}}{\emptyset F_{nv}}\right) f_{rv} \leq F_{nt}$$

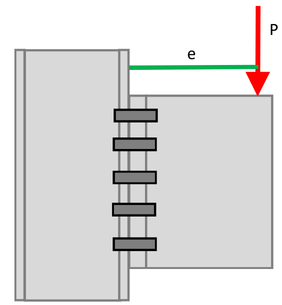
New Design Tension Strength:

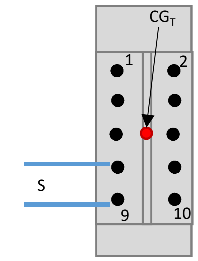
$\emptyset R_n = 0.75 F'_{nt} A_b$

$f_t(A_b) < \emptyset R_n$ ➔ Good

Bolted Connection Design – AISC Spec J

Combined Shear & Tension in Bearing Connections: Bracket Case II





Combined Shear & Tension

- AISC Sec. J3.7 – Combined V&T in Bearing Connections
- AISC Sec. J3.9 – Combined V&T in Slip Crit. Connections

When the required stress in either shear or tension is less the or equal to 30% of the available stress, the effects of combined stress need not be investigated.

Available tensile strength of bolts w/ tension & shear (R_n)

$R_n = F'_{nt} A_b$ $\qquad \emptyset R_n = 0.75 R_n$ for LRFD

Where: $F'_{nt} = 1.3 F_{nt} - \left(\frac{F_{nt}}{\emptyset F_{nv}}\right) f_{rv} \leq F_{nt}$

$F_{nt} = nominal\ tensile\ stress\ from\ AISC\ Table\ J3.2$

$F_{nv} = nominal\ shear\ stress\ from\ AISC\ Table\ J3.2$

$f_{rv} = required\ shear\ stress\ using\ LRFD$

A_b = nominal unthreaded body area of bolt or threaded part

Method of Design

AISC Ch 7
Case I (N.A. not @ C.G) or II (N.A. @ C.G.)

AISC Ch. 7 Case II

Shear force Per Bolt:

$r_{uv} = \frac{P_u}{n}$

Moment Effect per Bolt:

1) $M_u = P_u e$

2) There is no compression block. Bolt get "compressive load". Not exceeding clamping force.

3) Establish I_x

$I_x = A_b \left(\sum d_y\right)^2$ Where: d_y= bolt distance to NA

4) Tensile force in worst case bolts

$r_{ut} = \left(\frac{P_u\, e\, c}{I_x}\right) A_b$

Shear & Tensile Stress in Bolts at 30% Threshold?:

$f_{rv} = \frac{r_{uv}}{A_b}$ $\qquad f_t = \frac{r_{ut}}{A_b}$

Is $f_{rv} \geq 0.3\ \emptyset F_{nv}$? $\qquad \emptyset = 0.75\ for\ both\ cases$

Is $f_t \geq 0.3\ \emptyset F_{nt}$?

If either exceed 30% ➔ Consider combined V&T.

If not ➔ Done.

Available Tensile Strength (Revisited for Combined V&T):

$$F'_{nt} = 1.3 F_{nt} - \left(\frac{F_{nt}}{\emptyset F_{nv}}\right) f_{rv} \leq F_{nt}$$

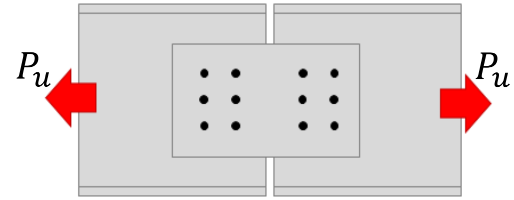
New Design Tension Strength:

$$\emptyset R_n = 0.75 F'_{nt} A_b$$

$f_t(A_b) < \emptyset R_n$ ➔ Good

Bolted Connection Design – AISC Spec J

Slip Critical Connections



If there is a shear load on the plates, a moment $\left(M = \frac{Re}{2}\right)$ must be considered. See Design Example II.A-20

Combined Shear & Tension

- AISC Sec. J3.8 – High Str. Bolts in Slip Crit. Connectoins
- AISC Sec. J3.9 – Combined V&T in Slip Crit. Connections
- AISC Sec. J3.10
- See Design Example II.A-20 for Moment Resolution for Shear Connections

Nominal/Available Slip Resistance of Connection (R_n) PER BOLT!

$R_n = \mu D_u h_f T_b n_s$

Where:

- D_u=1.13, a multiplier of bolt pretension to spec min pretension. Lower values approved by EOR.
- h_f = factor for fillers
  - No fillers or bolts added to distribute loads in filler: $h_f = 1.0$
  - Bolts NOT added to distribute loads in filler:
    - One filler btw connected parts: $h_f = 1.0$
    - Two or more fillers btw connected parts: h_f =0.85
- T_b= minimum fastener tension in **Table J3.1** kips
- n_s= number of slip planes
- μ = mean slip coefficient 0.3 for Class A faying surface. 0.5 for Class B

The Reduction Factor Depends on the Holes

- For Standard & short-slotted holes perpendicular to load direction
  - $\emptyset = 1.00\ R_n$
- For Oversized & short-slotted holes parallel to load direction
  - $\emptyset = 0.85\ R_n$
- For Long-Slotted Holes
  - $\emptyset = 0.70\ R_n$
- Finger shims up to ¼" are allowed per AISC J3.2.

Part 1: Identify Design Loads

Identify Initial Parameters

$\mu, D_u, h_f, T_b, n_s, \emptyset$

Nominal/Available Slip Resistance of Connection (R_n) PER BOLT!

$\emptyset R_n = \emptyset \mu D_u h_f T_b n_s$ ➔ $\#_{bolts} = \frac{P_u}{\emptyset R_n}$

Part 2: Check Remaining Parameters

Gross Section Yield

Check each piece!

$\emptyset_t T_n = \emptyset_t A_g F_y$ $\quad \emptyset_t$=0.9

Net Section Rupture

Check each piece!

$\emptyset_t T_n = \emptyset_t A_e F_u$ $\quad \emptyset_t$=0.75

Be sure to consider shear lag. $A_e = U A_n$

In the case of this plate, U=1.0 for full contact. **However, use U=0.85 because J4.1 specs it for splice plates!!!!**

Check Bolts in Shear

AISC Table 7-1 ➔ $\emptyset_v r_n$ in kips/bolt

$\emptyset_v R_n = n \emptyset_v r_n$ $\quad$ n = # of bolts in shear

Bolt Bearing Capacity

Shortcut Used Here: See 1st Bolting Slide for Methods

Use long way if shortcut won't work based on spacing

Check each piece (although it may be obvious that one governs based on thickness)!

AISC Table 7-4 or 7-5 ➔ $\emptyset_v r_n = \frac{bolt\ kips}{1\ inch\ of\ mtrl\ thickness}$

$\emptyset_v R_n = n[(\emptyset_v r_n)(thickness_{Thinnest\ mtrl})]$

n = # of bolts in shear

Block Shear: [AISC EQ J4-5]

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

U_{bs} = 1 when tension stress is uniform, 0.85 otherwise

$A_{nv} = nominal\ area\ subject\ to\ shear = A_{gv} - A_{hv}$

$A_{nt} = nominal\ area\ subject\ to\ tension$

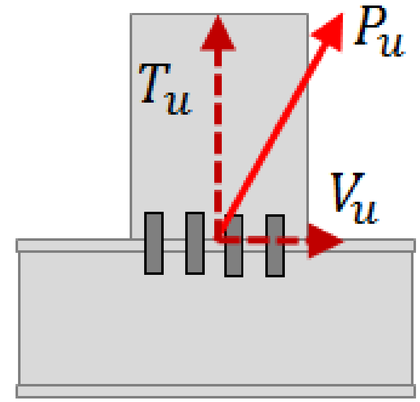
$\emptyset R_n = 0.75 R_n$

Failure Mode	LRFD (Kips)
Gross Section Yield	#
Net Section Rupture	#
Bolts in Shear	#
Bolt Bearing Capacity	#
Block Shear Rupture	#

Lowest Value GOVERNS

Bolted Connection Design – AISC Spec J

Combined Shear & Tension in **Slip Critical** Connections



Combined Shear & Tension
- AISC Sec. J3.7 – Combined V&T in Bearing Connections
- AISC Sec. J3.9 – Combined V&T in Slip Crit. Connections

When a slip-critical connection is subject to tension that reduces the net clamping force, the available slip resistance per bolt shall be multiplied by the factor k_{sc}

- $k_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$ (LRFD)
  - Where: T_u = required tension force using LRFD
    n_b =number of bolts carrying the applied tension
    $D_u = 1.13$
    T_b = min. fastener tension AISC Table J3.1

The Reduction Factor Depends on the Holes
- For Standard & short-slotted holes perpendicular to load direction
  - $\emptyset = 1.00\ R_n$
- For Oversized & short-slotted holes parallel to load direction
  - $\emptyset = 0.85\ R_n$
- For Long-Slotted Holes
  - $\emptyset = 0.70\ R_n$
- Finger shims up to ¼" are allowed per AISC J3.2.

Part 1: Identify Design Loads

Identify Initial Parameters
$\mu, D_u, h_f, T_b, n_s, \emptyset$ & n_b

Convert P_u Into Shear (T_u) & Tension (V_u) Vectors

Available Tensile Bolt Strength (AISC J3.6)
$R_n = F_{nt} A_b$ $with\ \emptyset = 0.75$ (J3.6) F_{nt} (Table J3.2)
Just use AISC Table 7-2
Check: $\emptyset R_n > \frac{T_u}{\#_{bolts}}$

Available Slip Resistance Per Bolt (if T_u $were$ 0)
$\emptyset R_{n1bolt} = \emptyset \mu D_u h_f T_b n_s$ (see previous slide)

Available Slip Resistance of the Connection
- Include Reduction Factor k_{sc} w/ $P_{ut} = T_u$
  - $k_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$
- $\emptyset R_{n(connection)} = (\emptyset R_{n1bolt})(k_{sc})(n_b)$

Check: $(\emptyset R_{n1bolt})(k_{sc})(n_b) > V_u$

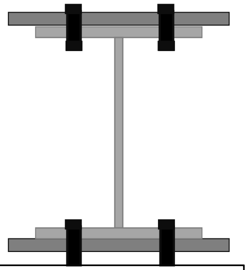
CHECK ALL ADDITIONAL LIMIT STATES FROM PREVIOUS SLIDE!!!!

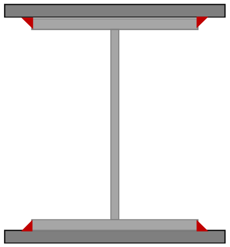
COVER PLATES FOR FLEXURE

Bolt: Textbook Ex. 12-3 Weld: Design Handout

Cover Plate Specification AISC F13.3

- Often included to resist bending deflection
- AISC F13.3 recommends using SC high-strength bolts or fillet welds to attach Cover Plate.
    - The attachment shall be strong enough to allow the cover plate to fully develop bending stresses. Commonly ➔ $\boldsymbol{M_{p(part)} = F_y Z_x}$
    - The cover plate must extend beyond the theoretical cutoff point (according to moment diagram) by a distance a' (when welding) or rule of thumb is "d" past critical zone.
        - **a' must develop required strength by the critical section.**
        - When there is a continuous weld with fillet size $\geq \frac{3}{4}\,plate\ thickness$ across plate ends ➔ $a'_{longitudinal}$= plate width (w)
        - A continuous weld smaller than $\frac{3}{4}(w)$ across end➔ $a'_{longitudinal}$=1.5(w)
        - When there is no weld across plate ends a'=2(w)
- Connections between beam flanges and cover plates need to handle longitudinal shear load.
    - Unit long. shear stress btw cover plate and flange ➔ $f_b = \frac{V_u Q}{Ib}\ \ {kip}/{in^2}$
    - Total shear force across flange for 1-inch of beam length ➔ $f_{b(design)} = \frac{V_u Q}{I}\ \ {kip}/{in}$
- Bolted cover plate not to exceed 70% of flange cross section area ➔$\frac{A_{cp}}{A_{cp}+A_{flange}} \leq 0.7$
- Bolt spacing not to exceed previously established limits (AISC Section E6.2)
    - Max Fastener Spacing ➔ $(Thickness_{Thinner\ Outside\ Plate})x\left(0.75\sqrt{\frac{E_{(Plate)}}{F_{y(plate)}}}\right) \leq 12in$
    - Or if bolts are staggered ➔ $(Thickness_{Thinner\ Outside\ Plate})x\left(1.12\sqrt{\frac{E_{(Plate)}}{F_{y(plate)}}}\right) \leq 18in$





Bolted Workflow:
Known V_u & $Bolt\ Type$

Check AISC Spec. F13.3
$\frac{A_{one\ cp}}{A_{one\ p} + A_{one\ flange}} \leq 0.7?$

Compute Shear Force
$\boldsymbol{f_{b(design)}} = \frac{\boldsymbol{V_u Q}}{\boldsymbol{I}}\ \ \boldsymbol{kip}/\boldsymbol{in}$
Parallel Axis Thm $\boldsymbol{I_x} = I_{beam} + \sum I_{pl} + A_{pl}D_{pl}^2$
I_x Includes both plates!
$Q = (A_{pl})x(D_{pl})$ ONLY 1 plate!

Bolt Strength $\emptyset = 0.75$

Bearing
$\emptyset \boldsymbol{R_n} = \emptyset\left(Lesser\ of \begin{cases}1.2l_c tF_u\\ 2.4d_b tF_u\end{cases}\right)(\#_{Involved\ bolts})$
(See bearing slide. Eqn may vary w/ hole type)

Shearing
$\emptyset \boldsymbol{R_n} = \emptyset(A_{bolt})(F_{ubolt})(\#_{Involved\ bolts})$ OR
$\emptyset \boldsymbol{R_n} = (Tbl\ 7-1\ value)(\#_{Involved\ bolts})$
Lower of Bearing & Shear Governs!

Bolt Spacing
$Spacing_{bolt} = \frac{\emptyset \boldsymbol{R_n}_{governing}}{\boldsymbol{f_{b(design)}}}\ \ \frac{kips}{kips/in} \Rightarrow in$

Checks
1) Spacing ≤ AISC Section E6.2 Max?
2) $L_{c(long)} \leq L_{c(edge)}$? So it won't affect bearing

Welded Workflow

Depending on the question:
EASY ANALYSIS PROBLEM
Either calculate the required strength by
$\boldsymbol{f_{b(design)}} = \frac{\boldsymbol{V_u Q}}{\boldsymbol{I}}\ \ \boldsymbol{kip}/\boldsymbol{in}$
with weld strength = 1.392 ${Kips}/{in}/_{16th}$

OR

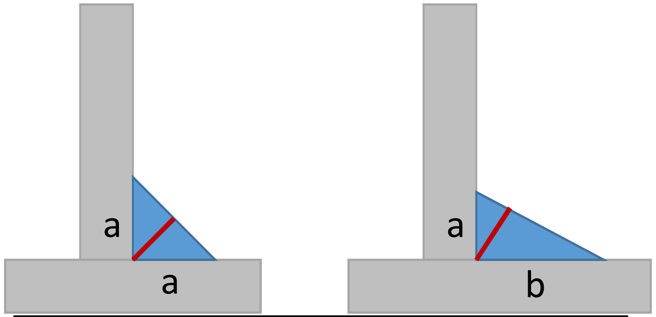
DIFFICULT DESIGN PROBLEM
Stipulations for a' and the need to fully develop the plastic moment of the added plates will require different weld strengths.
$Force = (F_y)(Distance_{from\ PNA})$
See *Alexander Newman* Design Example

Welded Connection Design – AISC Spec J

General Welding Information

Welding Information

- AISC Spec. J1 – General provisions for connection design
- AISC Spec. J2 – Welds
- AISC Spec. K – HSS & Box Member Connections
- AISC Part 8 – Design Considerations for Welds
- AISC Part 10 – Design of Simple Shear Connections
- AISC Part 11 – Design of Partially Restrained Moment Connections
- AISC Part 12 – Design of Fully Restrained Moment Connections
- AWS D1.1 – Structural Welding Code Steel



Equal leg fillet welds: $Throat_{eff} = 0.707a$

Unequal leg fillet welds: $Throat_{eff} = \frac{ab}{\sqrt{a^2+b^2}}$

Fillet Weld Size (in)	Number of Rod Passes
3/16	1
1/4	1
5/16	1
3/8	3
7/16	4
1/2	4
5/8	6
3/4	8

Table J2.4

Material Thickness of Thinner Part (in)	Minimum Size (in)
X ≤ 1/4	1/8
1/4 < X ≤ 1/2	3/16
1/2 < X ≤ 3/4	1/4
3/4 < X	5/16

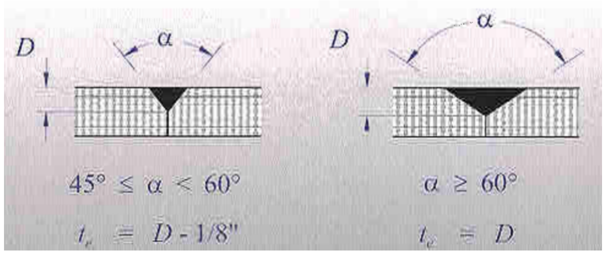
Tables

- AISC Table J2.1 – Effective Throat of PJP Groove Welds
- AISC Table J2.2 – Effective Throat of Flare Groove Welds
- AISC Table J2.3 – Min. Effective Throat of PJP Groove Welds
- AISC Table J2.4 – Min. Size of Fillet Welds
- **AISC Table J2.5 – ∅ values and Available Strength of Welded Joints**
- AISC Table 8-1 – Coefficient, C, for Conc. Loaded Weld Groups
- AISC Table 8-2 – Prequalified Welded Joints
  - Root tolerances, backing, etc.. From AWS.
- AISC Table 8-3 – Electrode Strength Coefficient, C_1
- AISC Table 8-4 to 8-11a – Coefficients, C, for Eccentric Loaded Weld Groups

Strength of PJP welds ➔ $t_e L(Weld\ Metal\ Strength)$

$\emptyset\ values\ and\ Weld\ Metal\ Strength\ taken\ from\ Table\ J2.5.$

t_e can be determined from Tables 8-2.



Welded Connection Design – AISC Spec J

General Welding Information

<u>Size Requirements for Fillet Welds</u>

- Effective Area = (effective length) x (effective throat)
- Minimum Size of Fillet Welds
  - **Table J2.4 – By material thickness of parts joined**
- Maximum Size of Fillet Welds
  - For mtrl. < ¼" thick ➔ Not greater than the mtrl. Thickness
  - For mtrl ≥ ¼" thick ➔ Not greater than the mtrl thickness – 1/16"
- Effective Length of fillet weld must be at least 4x nominal size.
  - (e.g. ¼" fillet weld must be 1" long) **or min of 1-1/2" for intermittent welds, whichever is greater**
- Minimum Thickness of Connected Elements
  - Base member in tension

$$t_{min} = \frac{0.6F_{EXX}0.707(w)}{F_{u(base)}}$$

  - Base member in shear

$$t_{min} = \frac{0.6F_{EXX}0.707(w)}{0.6F_{u(base)}} \rightarrow \frac{F_{EXX}0.707(w)}{F_{u(base)}}$$

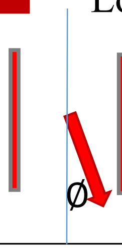
- Intermittent Fillet Welds
  - Minimum length is larger of **4x nominal size** or 1-1/2"
  - Built-up tension members ➔ max spacing is 300xr_s
    - Where r_s =radius of gyration of smaller member being welded AISC D4
  - Built-up compression members ➔ max spacing is
    - To connect two rolled shapes = 24 in
    - See AISC Spec E6.2

<u>Weld Strength</u>

- Function of a) base metal, b) weld metal, c) welding process, & d) weld penetration
- Nominal resistance of a weld (R_n)
  - For Base Metal $R_n = F_{n,BM}A_{BM}$
  - For Weld Metal $R_n = F_{nw}A_{We}$
  - For Tensile Members ➔ $R_{ne} = UF_{nw}A_{we}$ Don't forget shear lag.
- Fillet Weld Strength
  - $F_w = 0.60F_{EXX}$
  - $V_n = t_eF_{nw}L = 0.707\ w\ 0.60F_{EXX}\ L = 0.707\left(\frac{1}{16}\ in\right)0.6\left(70\frac{k}{in}\right)1in$
  - Per inch of fillet weld ➔ 1.86 k/in/16th
  - $\emptyset V_n = 0.75\left(\frac{1.86\frac{k}{in}}{16th}\right) = \frac{1.392\frac{k}{in}}{16th}$ ASD➔ $\frac{V_n}{\Omega} = \frac{1.86}{2} = \frac{0.928\frac{k}{in}}{16th}$
  - When the length (L) is over 100 times the leg size (w), multiply (L) by β
    - $\beta = 1.2 - 0.002(^L/_w) \leq 1.0$
    - If weld length is over 300w, use 180w.
- Necessary Tables
  - Table J2.1 – Effective throat of PJP welds
  - Table J2.2 – Effective Weld Sizes of Flare Groove Welds (common w/ HSS)
  - Table J2.3 – Minimum Eff. Throat Thickness of PJP groove welds
  - **Table J2.5 – Available Strength of Welded Joints (Very Important!!)**
- Limitations
  - Longitudinal fillet weld lengths may not be less than the distance between them because of **Shear Lag** (Case 4 Table D3.1).
  - In Lap Joints, minimum amount of lap permitted is 5x thickness of thinner part, but not less than 1 inch.

Welded Connection Design – AISC Spec J

Loaded Fillet Welds

A) 

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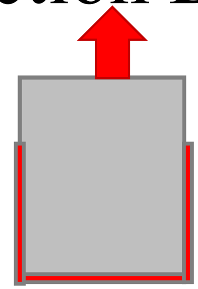
C) 

<u>Transverse/Angled Loading of Fillet Welds</u>

- J2.4 allows transverse loading allows an increase of strength
  - A) For a linear weld group w/ uniform leg size, loaded through center of gravity,

    $R_n = F_{nw}A_{we}$ w/ $\emptyset = 0.75$

    $F_{nw} = 0.60F_{EXX}(1.0 + 0.50sin^{1.5}\theta)$

    θ measured off longitudinal axis.
  - B) For weld elements analyzed w/ ICR, R_{nx} , R_{ny} , & M_n are permitted to be determined as given on AISC pg 16.1-116
  - C) Fillet weld groups concentrically loaded w/ longitudinal and transverse elements. R_n= greater of: w/ $\emptyset = 0.75$

    1. $R_n = R_{nwl} + R_{nwt}$

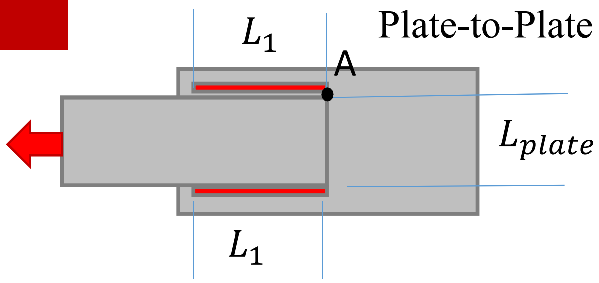
    Or

    2. $R_n = 0.85R_{nwl} + 1.5R_{nwt}$

    Note: Don't use $F_{nw} = 0.60F_{EXX}(1.0 + 0.50sin^{1.5}\theta)$ for these cases b/c transverse & longitudinal fillet welds have different deformation properties and limits. Use $R_{nwl}\&R_{nwt} = F_wA_w$

    F_w ➔ commonly $0.60F_{EXX}$ & A_w ➔ $(0.707)w(Length)$

Welded Connection Design – AISC Spec J



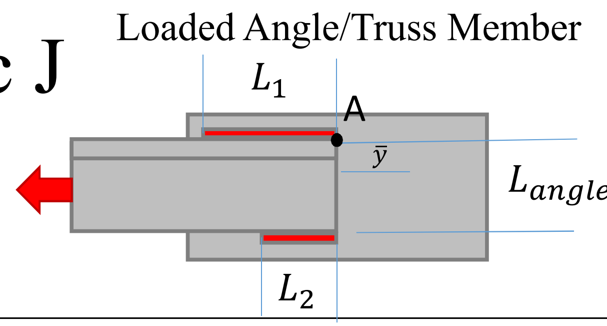
U to Lengthen Welds @ end (preferred b/c accurate U factor)

$R_u = lower\ of\ \emptyset P_n = \emptyset F_y A_g$ w/$\emptyset = 0.9$

Select Fillet Weld Size & total length L_w assuming U=1.0 $L_w = 2\ L_1$

Shear Lag Factor from Case 4

$$L_{1,req} = \frac{L_{1,calc}}{U}$$



Angle to Plate Welded Connections

- Prescribing welds balanced about the neutral axis
  - Weld group centroid coincides w/ centroid of tensile load.

Method 1: Shear Lag Up Front

Nominal Tensile Capacity of member with lower A_g

$R_u = lower\ of\ \emptyset P_n = \emptyset F_y A_g$ w/$\emptyset = 0.9$

$\emptyset P_n = \emptyset F_u A_e$ w/$\emptyset = 0.75$ & $A_e = UA_n$

Consider shear lag from Table 4.1 case 2 or 4. Some apply U=0.87 up front (case 4)

Select Fillet Weld Size & total length L_w

Take Moment about "Point A"

$$R_u(\bar{y}) - L_{angle}P_2 = 0$$

$$L_{2,calc} = \left(\frac{P_2}{Weld\ Str.\ per\ inch}\right)$$

$$L_{1,calc} = L_w - L_{2,calc}$$

Block Shear:

$R_n = 0.6F_u A_{nv} + U_{bs}F_u A_{nt} \le 0.6F_y A_{gv} + U_{bs}F_u A_{nt}$ [AISC J4-5]

$U_{bs} = 1$ when tension stress is uniform, 0.85 otherwise

$A_{nv} = nominal\ area\ subject\ to\ shear = (L_w)t_{base}$

$A_{nt} = nominal\ area\ subject\ to\ tension = (L_{angle\ leg})(t_{base})$

$\boldsymbol{\emptyset R_n = 0.75R_n \ge R_u}$ ***calc'd in step 1 to find weld length***

Method 2: Shear Lag to Lengthen Welds at the end (preferred b/c accurate U factor)

$R_u = lower\ of\ \emptyset P_n = \emptyset F_y A_g$ w/$\emptyset = 0.9$

Select Fillet Weld Size & total length L_w assuming U=1.0

Take Moment about "Point A"

$$R_u(\bar{y}) - L_{angle}P_2 = 0$$

$$L_{2,calc} = \left(\frac{P_2}{Weld\ Str.\ per\ inch}\right)$$

$$L_{1,calc} = L_w - L_{2,calc}$$

Shear Lag Factor Higher of Case 2 or 4 (Case 2 & 4 evaluated w/ longer weld)

$$L_{1,req} = \frac{L_{1,calc}}{U} \qquad L_{2,req} = \frac{L_{2,calc}}{U}$$

Welded Connection Design – AISC Spec J

Eccentrically Loaded Weld Groups

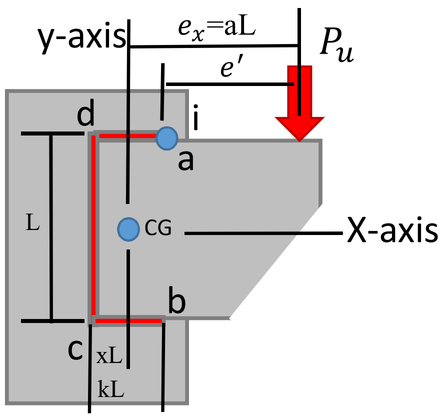
Eccentrically Loaded Weld Groups
- Two commonly used methods
  - Elastic Method
  - Instantaneous Center of Rotation Method

Elastic Method
- Very Conservative
- More simplistic method when deign aids are unavailable
- Statics-based
- Pieces welded together are assumed to be completely rigid
  - All deformation is in the weld
- Based on a summation of forces in 3-D space from P_{xyz} & M_{xyz}
- There are two ways to approach this method
  - 1) **Express/simplified methods** - for weld group shapes where location of max stress is known.
  - 2) **General method** – intensive hand calc, but applicable to all planar weld groups

ICR Method Ultimate Strength Method
- More accurate
- AISC Table 8-3 to 8-11 can be used to aid the design
- Nominal strength of weld group ➔ $R_{n,kips} = CC_1DL_{in}$ w/ $\emptyset = 0.75$
  - C = coefficient from AISC Table 8-#
  - C_1= electrode coefficient from AISC Table 8-3 $(C_1 = 1.0\ for\ F_{EXX})$
  - D = number of 1/16ths of an inch of fillet weld size
  - L_{in}= characteristic length of weld group in inches
- Available strength must be greater than required strength
  - $P_u \leq \emptyset R_n$ ➔ $P_u \leq \emptyset CC_1DL_{in}$
- Minimum required values for C, D, & L are given in the Tables 8-#
  - $C_{min} = \frac{P_u}{\emptyset C_1DL}$ $D_{min} = \frac{P_u}{\emptyset C_1CL}$ $L_{min} = \frac{P_u}{\emptyset C_1CD}$

Welded Connection Design – AISC Spec J

Welded Bracket w/ Eccentric Shear



<u>Express/Simplified Elastic Method Workflow (Max stress assumed @ point i): Size of required fillet weld</u>

- Variables for Elastic Method

Total weld length: L_{wt}
Centroid Location (xL): use AISC Table 8-# for this if needed
MOI about X&Y-axis& Polar w/ unit weld lengths:

$I_x = \sum \frac{bh^3}{12} + Ad^2 \rightarrow \frac{L^3}{12} + 2(KL)\left(\frac{L}{2}\right)^2$

$I_y = \sum \frac{bh^3}{12} + Ad^2 \rightarrow 2\frac{(KL)^3}{12} + 2(KL)\left(\frac{KL}{2} - (xL)\right)^2 + L(xL)^2$

J = $I_0 = I_x + I_y$

- Forces @ points a & b

$f_h = \frac{Tv}{J} \rightarrow \frac{((P_u)(e_x))\frac{L}{2}}{J}$ $\quad f_v = \frac{Th}{J} \rightarrow \frac{((P_u)(e_x))((KL)-(xL))}{J}$ $\quad f_s = \frac{P_u}{L_{wt}}$ $\quad f_r = \sqrt{f_h^2 + f_v^2 + f_s^2}$

Required weld size = $\frac{f_r\,(\frac{kips}{inch})}{1.392\frac{kips}{in}/sixteenth}$ = ______ sixteenths of fillet weld. Check vs AISC Table J2-4

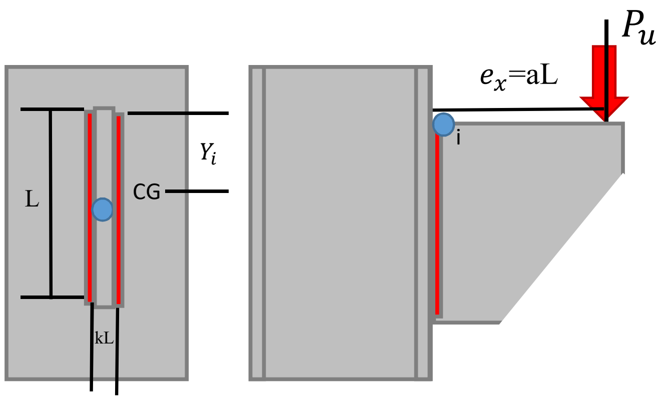
<u>ICR Method Workflow</u>: Use AISC Tables 8-5 to 8-11 depending on the shape of the weld group

A) Givens: L, e', kL ➔ Determine required fillet weld size
Infer: $k = \frac{kL}{L}$ ➔ Table 8-# for "X" coefficient ➔ "xL" to get e_x ➔ e_x to get a = $\frac{e_x}{L}$ ➔ C from table
Solution: $D_{\min 1/16ths} = \frac{P_u}{\emptyset C_1 CL}$ compare w/ AISC Table J2.4 mtrl thickness req.

B) Givens: e_x , L ➔ Determine kL & required fillet weld size (D # of sixteenths)
Infer: Select initial D from steel thicknesses➔ $C_{min} = \frac{P_u}{\emptyset C_1 DL}$ ➔ k from Table 8-# ➔ kL for that "D"
Solution: $D_{\min 1/16ths} = \frac{P_u}{\emptyset C_1 CL}$ com

Welded Connection Design – AISC Spec J

Welded Group Eccentrically Loaded Normal to Faying Surface



Express/Simplified Elastic Method Workflow (Max stress assumed @ point i): Size of required fillet weld

- Variables for Elastic Method

Total weld length: L_{wt} $\quad (L_{wt} = 2L \quad in\ this\ case)$

MOI about X-axis w/ unit weld lengths:

$I_x = \sum \frac{bh^3}{12} \rightarrow 2\frac{L^3}{12}$

- Forces @ point I

$f_v = \frac{P_u}{L_{wt}} \qquad f_h = \frac{P_u\, e\, Y_i}{I_x} \qquad f_r = \sqrt{f_h^2 + f_v^2} \quad \frac{kips}{inch}$

Required weld size = $\frac{f_r\, (\frac{kips}{inch})}{1.392\frac{kips}{in}/sixteenth}$ = ______ sixteenths of fillet weld. Check vs AISC Table J2-4

ICR Method Workflow:

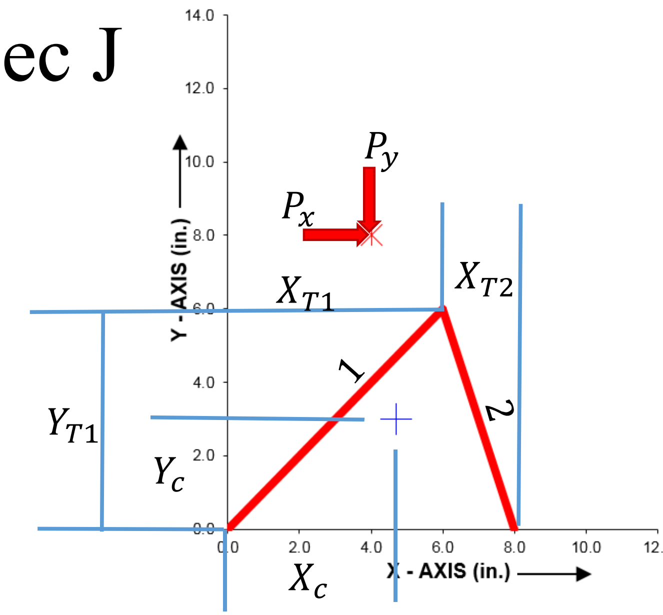
A) Givens: e_x , L ➔ Determine required fillet weld size (D # of sixteenths)

Infer: Assumption that K=0 (not ideal) , a= $\frac{e_x}{L}$

Solution: $D_{1/16ths} = \frac{P_u}{\emptyset C_1 CL}$ = ________ sixteenths of fillet weld. Check vs AISC Table J2-4

Welded Connection Design – AISC Spec J

General Elastic Method for Any Weld Group w/ Same Size Welds



WELD GROUP PLOT

General Elastic Method (as adopted from Alex Tomanovich)

1) Identify Weld Group Main Parameters

- L_{wt} = total length of weld group
- Centroid of Weld Group
  - $X_c = \frac{\sum X_c}{L_{wt}}$ when $X_c = (X_{mid,indv})(L_{w,indv})$
  - $Y_c = \frac{\sum Y_c}{L_{wt}}$ when $Y_c = (Y_{mid,indv})(L_{w,indv})$
- Calculate I_x, I_y, J
  - $I_x = (\sum I_{x0}) - L_{wt}(Y_c)^2 \rightarrow I_{xo1} = \frac{(L_1)(Y_{T\,1})^2}{12} + L_1\left(Y_{mid,1}\right)^2$ continue for each weld
  - $I_y = (\sum I_{y0}) - L_{wt}(X_c)^2 \rightarrow I_{yo1} = \frac{(L_1)(X_{T\,1})^2}{12} + L_1\left(X_{mid,1}\right)^2$ continue for each weld
  - $J = I_x + I_y$

2a) Moment Summations about CG (for each point loaded)

- **USE RIGHT HAND RULE CENTERED ON THE CG_{weld}!!!!**

$M_{xfrom(P_y)} = \left(Z_{load\ point} - Z_{CG_{weld}}\right)(P_y)$

$M_{xfrom(P_z)} = \left(Y_{load\ point} - Y_{CG_{weld}}\right)(P_z)$

$M_{yfrom(P_x)} = \left(Z_{load\ point} - Z_{CG_{weld}}\right)(P_x)$

$M_{yfrom(P_z)} = \left(X_{load\ point} - X_{CG_{weld}}\right)(P_z)$

$M_{zfrom(P_x)} = \left(Y_{load\ point} - Y_{CG_{weld}}\right)(P_x)$

$M_{zfrom(P_y)} = \left(X_{load\ point} - X_{CG_{weld}}\right)(P_y)$

2b) Loads Transformed for Effects @ CG of Weld Group

Notation $M_{xfrom(P_y)}$ indicates moment about x-axis caused by P_y.

- $\sum P_x = Applied\ P_x$
- $\sum P_y = Applied\ P_y$
- $\sum P_z = Applied\ P_z$
- $\sum M_x = Applied M_x + M_{xfrom(P_y)} + M_{xfrom(P_z)}$
- $\sum M_y = Applied M_y + M_{yfrom(P_x)} + M_{yfrom(P_z)}$
- $\sum M_z = Applied M_z + M_{zfrom(P_x)} + M_{zfrom(P_y)}$

Welded Connection Design – AISC Spec J

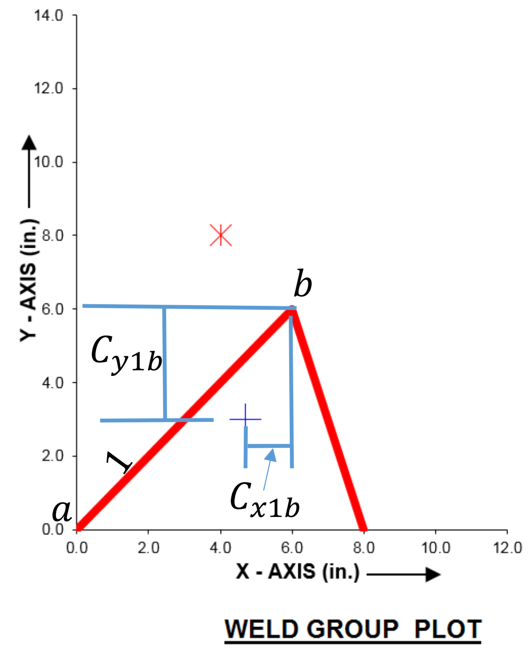
Elastic Method General Algorithm

General Elastic Method (as adopted from Alex Tomanovich)

3) Weld Forces @ Individual Points (Weld Ends): The forces are dependent on Cx & Cy, as measured from CG_{weld}

BE AWARE OF THE MOMENT ORIENTATION AND RIGHT HAND RULE!!!

- $f_z = \frac{\sum P_z}{L_{wt}} + \frac{(\sum M_x)\,(C_{y1b})}{I_x} + \frac{(\sum M_y)\,(C_{x1b})}{I_y}$
- $f_{vy} = \frac{\sum P_y}{L_{wt}} + \frac{(\sum M_z)\,(C_{x1b})}{J}$
- $f_{vx} = \frac{\sum P_x}{L_{wt}} + \frac{(\sum M_z)\,(C_{y1b})}{J}$ (e.g. positive moment about Z will produce force in negative X if C_{y1b} is positive)
- $f_{resultant} = \sqrt{(f_z)^2 + (f_{vy})^2 + (f_{vx})^2}$ $\frac{Force}{Unit\ Length}$ ➔ Usually $\frac{Kips}{in}$



4) Weld Requirements and Base Metal Check

- Fillet weld size required (in 16ths of an inch) = $\frac{f_{resultant}}{0.75*w*(0.6F_{EXX})*(unit\ length)} = \frac{f_{resultant}}{1.392\ ^k/_{in}/_{16th}}$ for LRFD.
  - If using ASD, substitute $0.928^k/_{in}/_{16th}$ for $1.392\ ^k/_{in}/_{16th}$
- Base Metal Shear Allowable
  - LRFD: $\emptyset R_n = (1.00)(0.6)(F_{ybase})(Base\ Metal\ Thickness)$ ➔ ______ $^{force}/_{unit\ length}$ usually $\frac{Kips}{inch}$
  - Refer to AISC Spec. J4
  - $R_n = 0.6F_yA_{gv}$ $\quad w/\ \emptyset = 1.00\ or\ \Omega = 1.5\ \ (This\ is\ where\ 0.4F_y\ for\ ASD\ comes\ from)$

Welded Connection Design – AISC Spec K

HSS connections

Welding Information

- AISC Spec. K1 – Concentrated Forces on HSS
- AISC Spec. K2 – HSS to HSS Truss Connections
- AISC Spec. K3 – HSS to HSS Moment Connections
- AISC Spec. K4 – Welds of Plates & Branches to Rectangular HSS

Tables

- AISC Table K1.1 – Available Strength of Plate-to-Round HSS Connections
- AISC Table K1.2 – Available Strength of Plate-to-Rectangle HSS Connections
- AISC Table K2.1 – Available Strengths of Round HSS-to-HSS Truss Connections
- AISC Table K2.2 – Available Strengths of Rectangular HSS-to-HSS Truss Connections
- AISC Table K3.1 – Available Strengths of Round HSS-to-HSS Moment Connections
- AISC Table K3.2 – Available Strengths of Rectangular HSS-to-HSS Moment Connections
- AISC Table K3.3 – Effective Weld Properties for Connections to Rectangular HSS

Note: Each table is followed by the applicable limit states. If all criterion can't be met, you can't use that particular table.

Method For Using Tables in AISC Spec. K To Determine Weld Sizes and Whether the HSS Must Be Reinforced

Check the Limits of Applicability in Table K#.#A

Compute Nominal Strength and Design Strength for given Connection type.

- Lowest Limit State Governs

Select welds based on the allowable load in previous step.

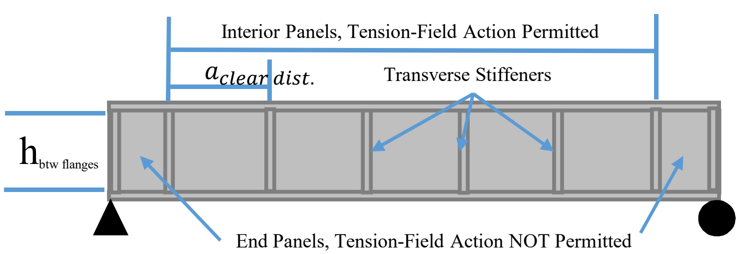
Sometimes CJPs are needed if the plate is small.

Plate Girder Design – AISC Spec F & G

AISC Specs
- AISC Spec F for Flexural Design
- AISC Spec G for Shear Design

General Information
- Regular Plate Girder: All steel has the same yield strength.
- Hybrid Girder: Flanges have a higher yield strength than web.
- May be stiffened or unstiffened
- Stiffened Girders may be designed for Tension Field Action


Unstiffened Plate Girder

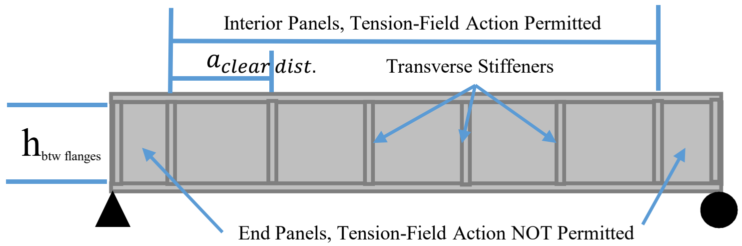


Stiffened Plate Girder

Plate Girder Proportioning Limits (AISC Spec F13)
- Holes in tension flange ➔ See F13.1 to check tensile rupture limit state
  - If $F_u A_{fn} \geq Y_t F_y A_{fg}$ ➔ tensile rupture doesn't apply
    Where Y_t = hole reduction coefficient = 1.0 for $F_y/F_u \leq 0.8$, 1.1 otherwise
  - If $F_u A_{fn} < Y_t F_y A_{fg}$ ➔ nominal flexural strength at holes is less than:
    $M_n = \frac{F_u A_{fn}}{A_{fg}} S_x$
- Proportion Limits for I-Shaped Members
  - For Singly Symmetrical I-Shaped Members
    $0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9$ where I_{yc} = moment of inertia of the Y-axis for comp flange.
- I-Shaped Members w/ slender webs $\frac{h}{t_w} > 5.7\sqrt{\frac{E}{F_y}}$ (AISC Table B4.1b Case 15) must be designed according to AISC Spec F5 & satisfy these additional limits:
  - Web thickness upper limit (when using slender web) ➔ $t_w < \frac{h}{5.7\sqrt{\frac{E}{F_y}}}$
  - For UNSTIFFENED girders ➔ $\frac{h}{t_w} \leq 260$ & $\frac{A_{web}}{A_{fc}} \leq 10$
  - For STIFFENED girders when $\frac{a}{h} \leq 1.5$ ➔ $\left(\frac{h}{t_w}\right)_{max} = 12\sqrt{\frac{E}{F_y}}$
    $\frac{a}{h} > 1.5$ ➔ $\left(\frac{h}{t_w}\right)_{max} = \frac{0.4E}{F_y}$

Plate Girder Design – AISC Spec F & G

Unstiffened Plate Girder



Stiffened Plate Girder

Compact, Noncompact, Slender? Table B4.1b Case 11 example.

Be careful with "b" dimension and "t"
For I shaped members use half of "b" for flange width

$\frac{b}{t}$ (or $\frac{b_f}{2t_f}$ for I shaped).

$\lambda = Value$ $\quad\lambda_{pf} = Value$ $\quad\lambda_{rf} = Value$

$Compact$: $\lambda < \lambda_{pf}$

$Noncompact$: $\lambda_{pf} < \lambda < \lambda_{rf}$

$Slender$: $\lambda_{rf} < \lambda$

$$S_{xt} = \frac{I_x}{c_{tf}}$$

$$S_{xc} = \frac{I_x}{c_{cf}}$$

<u>Plate Girder Flexural Design Strength</u>

- Consider 4 limit states (AISC F5)

1) Compression flange yielding

$$M_n = R_{pg} F_y S_{xc} \qquad R_{pg} = 1 - \left(\frac{a_w}{1200-300a_w}\right)\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \leq 10.0$$

2) Lateral-torsional buckling

- $M_n = R_{pg} F_{cr} S_{xc}$ $\quad F_{cr}$ depends on L_b, $L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$, & $L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}}$

where $r_t = \dfrac{b_{fc}}{\sqrt{12\left(\frac{h_o}{d} + \left(\frac{a_w}{6}\right)\left(\frac{h^2}{h_o d}\right)\right)}}$ $\quad h_o$=distance btw flange centroids

$L_b \leq L_p$	$L_p \leq L_b \leq L_r$	$L_r \leq L_b$
LTB Doesn't Apply	$F_{cr} = C_b\left(F_y - 0.3F_y\left(\frac{L_b - L_p}{L_r - L_p}\right)\right) \leq F_y$	$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \leq F_y$

3) Compression flange local buckling

- $M_n = R_{pg} F_{cr} S_{xc}$

$$K_c = 0.35 \leq \frac{4}{\sqrt{\frac{h}{t_w}}} \leq 0.76$$

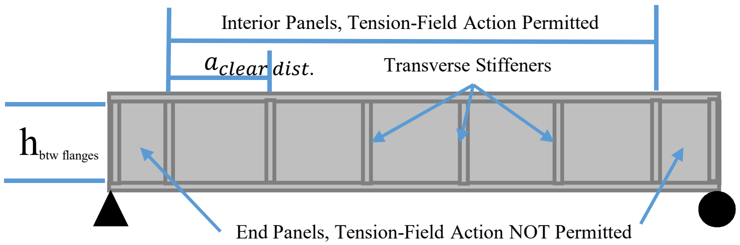
$Compact$ $Flange$	$NonCompact$ Flange	$Slender$ $Flange$
CFLB Doesn't Apply	$F_{cr} = \left(F_y - 0.3F_y\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right)$	$F_{cr} = \frac{0.9EK_c}{\left(\frac{b_f}{2_{tf}}\right)^2}$

4) Tension flange yielding

- $M_n = F_y S_{xt}$ $\quad$ Only applies when $S_{xt} < S_{xc}$ $\quad$ When $S_{xt} \geq S_{xc}$ TFY doesn't apply

Plate Girder Design – AISC Spec F & G

Unstiffened Plate Girder



Stiffened Plate Girder

<u>Tension Field Action (AISC Sect. G3)</u>

- TFA Permitted for flanged members when the web plate is supported on all four sides by flanges or stiffeners.
- Not permitted (meaning AISC Sec G2 Governs):
    - In end panels of members with transverse stiffeners
    - When a/h exceeds 3.0 or $\left[\frac{260}{\left(\frac{h}{t_w}\right)}\right]^2$
    - When $\frac{2A_w}{(A_{fc}+A_{ft})} > 2.5$
    - When $\frac{h}{b_{fc}}$ or $\frac{h}{b_{ft}} > 6.0$
- Available shear strength is a function of a/h. Tables for $\emptyset V_n/A_w$
    - AISC Table 3-16a ($F_y = 36ksi, TFA\ not\ included$)
    - AISC Table 3-16b ($F_y = 36ksi, TFA\ included$)
    - AISC Table 3-17a ($F_y = 50ksi, TFA\ not\ included$)
    - AISC Table 3-17b ($F_y = 50ksi, TFA\ included$)

<u>Plate Girder Shear Strength ➔ AISC G2 or G3 Depending on TFA Inclusion</u>

- Tension Field Action Prohibited $\emptyset = 0.9$
    - $V_n = 0.6F_yA_wC_v$ AISC Eq G2-1

C_v ➔when: ${}^{h}/_{t_w} \leq 1.10\sqrt{{}^{k_vE}/_{F_y}}$ ➔ $C_v = 1.0$

$1.10\sqrt{{}^{k_vE}/_{F_y}} \leq {}^{h}/_{t_w} \leq 1.37\sqrt{{}^{k_vE}/_{F_y}}$ ➔ $C_v = \frac{1.10\sqrt{{}^{k_vE}/_{F_y}}}{{}^{h}/_{t_w}}$

${}^{h}/_{t_w} \leq 1.37\sqrt{{}^{k_vE}/_{F_y}}$ ➔ $C_v = \frac{1.51k_vE}{\left(\frac{h}{t_w}\right)^2 F_y}$

k_v ➔ $k_v = 5.0$ for unstiffened webs with ${}^{h}/_{t_w} < 260$

stiffened webs with a/h > 3.0 or a/h > $(260/(h/t_w))^2$

1.2 for stem of tee shapes.

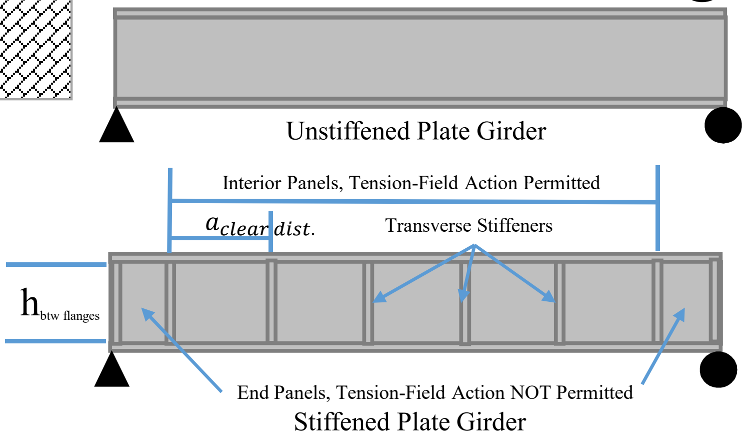
For webs with transeverse stiffensers $5 + \frac{5}{(a/h)^2}$

- Tension Field Action Permitted $\emptyset = 0.9$ (k_v & C_v calculated as above)

When: ${}^{h}/_{t_w} \leq 1.10\sqrt{{}^{k_vE}/_{F_y}}$ ➔ $V_n = 0.6F_yA_w$

${}^{h}/_{t_w} > 1.10\sqrt{{}^{k_vE}/_{F_y}}$ ➔ $V_n = 0.6F_yA_w\left(C_v + \frac{1-C_v}{1.15\sqrt{1+\left({}^{a}/_{h}\right)^2}}\right)$

Plate Girder Design – AISC Spec F & G



<u>Transverse Stiffeners w/out TFA (AISC G2-2)</u>

- Minimum moment of inertia $I_{st} \geq b t_w^3 j$
  Where: b = smallest of a & h; $j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5$
- Transverse stiffeners may be terminated short of the tension flange as long as BEARING is not needed to transmit concentrated loads (see ORANGE section).
- Stiffener-to-web welds should be terminated between $4t_w$ & $6t_w$ from the near toe of the web to flange weld. When single stiffeners are used, they shall be attached to the compression flange.
  - Bolts connecting stiffeners shall be spaced no more than 12 in on center
  - Intermittent field welds shall not be more than $16t_w$ clear distance and less than 10"

<u>Transverse Stiffeners General AISC Sec G2-2 & 2-3</u>

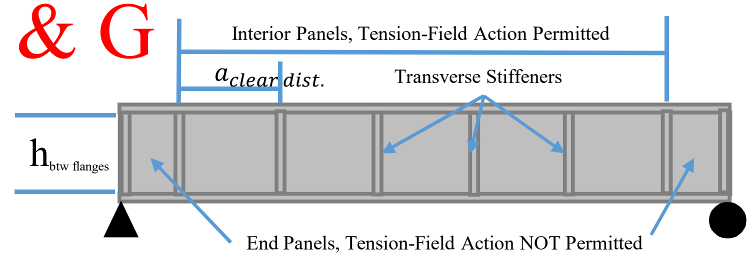
- **NOT** required when
  - $h/t_w < 260$
  - $h/t_w \leq 2.46\sqrt{\frac{E}{F_y}}$
  - Available design strength greater than shear load
    - $\emptyset V_n > V_u$ when $k_v = 5.0$

<u>Transverse Stiffeners w/ TFA (AISC G2-3)</u>

- These requirements are in addition to those required for Trans Stiff w/out TFA (above)
- $\left(\frac{b}{t}\right)_{st} \leq 0.56\sqrt{\frac{E}{F_{y,st}}}$ where: $\left(\frac{b}{t}\right)_{st}$ is the width-to-thickness ratio of the stiffener
  $F_{y,st}$ is the specified minimum yield stress of stiffener
- $I_{st} \geq I_{st1} + (I_{st2} - I_{st1})\left(\frac{V_r - V_{c1}}{V_{c2} - V_{c1}}\right)$ where: V_r is the larger of V_u in adjacent web panels.
  V_{c1} = smaller available V_n in adjacent web panels. Calculated using AISC G2-1
  V_{c2} = smaller available V_n in adjacent web panels. Calculated using AISC G3-2.
  I_{st1} = min. moment of inertia AISC G2-2 (above eq.)
  I_{st2} = min. moment of inertia of trans. Stiffener required to develop full web shear buckling plus web tension field resistance: $\frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$
  Where: ρ_{st} = larger of F_{yw}/F_{yst} and 1.0

Plate Girder Design – AISC Spec F & G

General Design Flowchart Pt 1



Stiffened Plate Girder

Read your damn textbook and PE review book or maybe the AASHTO comprehensive design example. It’s a bitch.

Composite Steel Members – AISC Spec I

Design of Composite Members
- AISC Spec. I1 – General Provisions
- AISC Spec. I2 – Axial Force
- AISC Spec. I3 – Flexure
- AISC Spec. I4 – Shear
- AISC Spec. I5 – Combined Axial & Flexure
- AISC Spec. I6 – Load Transfer
- AISC Spec. I7 – Composite Diaphragms & Collector Beams
- AISC Spec. I8 – Steel Anchors
- AISC Spec. I9 – Special Cases

The AISC Manual Includes the Following Composite Members:
- Steel Axial Compression Members
  - Steel members fully encased in concrete
  - HSS filled with concrete
- Steel Flexural Members
  - Steel members fully encased in concrete
  - HSS filled with concrete
- Steel beams anchored to concrete slabs so that they act together to resist bending
  - Concrete resists compression and steel resists tension

AISC Design Methods
- Two Main Design Methods
  - Plastic Stress Distribution Method
    - Steel components assumed to reach F_{yc} or F_{yt} while concrete components are assumed to reach a compressive stress of $0.85f_c'$ ($0.95f_c'$ for HSS filled with concrete).
  - Strain-Compatibility Method
    - Based on linear distribution of strain in cross section.
    - Max. concrete compressive strain = 0.003 in/in.

Composite Steel Members – AISC Spec I

AISC Spec. I1 – General Provisions

- Concrete design shall comply with ACI 318 with the following limits/exceptions:
  - ACI 318 Sections 7.8.2 and 10.13, and Ch. 21 shall be excluded.
  - Concrete & Steel reinforcement shall be specified in Section I1.3
  - Transverse Reinforcement limits shall be specified in Section I2.1a(2) in addition to those specified in ACI 318.
  - Minimum longitudinal reinforcement ratio for encased composite members shall be specified in I2.1a(3).
  - Use LRFD
- Material Limitations
  - Normal wt. concrete ➔ 3 ksi $\leq f_c' \leq$ 10 ksi
  - Lt. wt. concrete ➔ 3 ksi $\leq f_c' \leq$ 6 ksi
  - For column strength, maximum steel F_y = 75 ksi
- Classification of Filled Sections for Local Buckling
  - For compression, filled composite sections can be 1) compact, 2) noncompact, or 3) slender
    - See Table I1.1a
  - For flexure, filled composite sections can be 1) compact, 2) noncompact, or 3) slender
    - See Table I1.1b
  - See Tables B4.1a & B4.1b for definitions of width and depth (b and D) and thickness (t) for rectangle and round HSS

Composite Steel Members – AISC Spec I

AISC Spec. I2 – Axial Force   Encased Composite Members

- Encased Composite Members (Steel core surrounded by concrete)
  - Limitations: 1) cross section area of steel core ≥ 1% total section
    2) Shall be reinforced w/ continuous longitudinal bars and lateral ties/spirals.
    3)Spacing of trans. reinf. shall be smallest of: a) half smallest member dimension
    b) 16xdiameter of longitudinal reinf.
    c) 48xdiameter of lateral reinf.
    4) AISC says lateral ties min. of #3 bar spaced at max 12” OC or #4 bar spaced max 16”OC. Max spacing of lateral ties shall not exceed 0.5 x least column dimension.
    5) Min Reinf. Ratio $\rho_{sr} = \frac{A_{sr}}{Ag} \geq 0.004$ where A_{sr}= area of continuous steel reinf

- Compressive Strength $\emptyset_c = 0.75$ $P_u \leq \emptyset_c P_n$

  P_n depends on “elastic buckling load” $\left(P_e = \frac{\pi^2 (EI)_{eff}}{(KL)^2}\right)$ & “nominal compressive strength of column, disregarding adjustment due to length” $\left(P_{no} = F_y A_s + F_{y,sr} A_{sr} + 0.85 f_c' A_c\right)$.

  $$(EI)_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \qquad C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \qquad E_{c,ksi} = \left(w_{c,pcf}\right)^{1.5} \sqrt{f'_{c,ksi}}$$

  When: $P_{no}/P_e \leq 2.25$ ➔ $P_n = 0.658^{\frac{P_{no}}{P_e}} P_{no}$

  $P_{no}/P_e \geq 2.25$ ➔ $P_n = 0.877 P_e$

- Tensile Strength $\emptyset_c = 0.90$ $P_u \leq \emptyset_t P_n$ (neglect the tension capacity of the concrete)

  $P_n = F_y A_s + F_{y,sr} A_{sr}$

Composite Steel Members – AISC Spec I

AISC Spec. I2 – Axial Force   Filled Composite Members

- Filled Composite Members (Concrete surrounded by steel)
  - Limitations: 1) Cross sectional area of steel section shall be at least 1% of total composite cross section
    2) Classified for local buckling in accordance w/ section I1.4.
    See Table I1.1A
  - Compressive Strength: Available compressive strength of doubly symmetric filled composite members $\emptyset_c = 0.75$ $P_u \leq \emptyset_c P_n$
    1) section I2.1b w/ following mods.
    2) Determine if section is compact, noncompact, or slender by Table I1.1A

Compact	*NonCompact*	*Slender*
$P_{n0} = P_p = F_y A_s + C_2 f_c' \left(A_c + A_{sr}\frac{E_s}{E_c}\right)$ For Rectangle HSS, $C_2 = 0.85, Round = 0.95$	$P_{n0} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2$ $P_y = F_y A_s + 0.7 f_c' \left(A_c + A_{sr}\frac{E_s}{E_c}\right)$	$P_{n0} = F_{cr} A_s + 0.7 f_c' \left(A_c + A_{sr}\frac{E_s}{E_c}\right)$ For rectangle $F_{cr} = \frac{9E_s}{\left(\frac{b}{t}\right)^2}$ For Round $F_{cr} = \frac{0.72\,F_y}{\left(\left(\frac{D}{t}\right)\frac{F_y}{E_s}\right)^{0.2}}$

3) $P_e = \frac{\pi^2 (EI)_{eff}}{(KL)^2}$ $(EI)_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c$ $C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9$ $E_{c,ksi} = \left(w_{c,pcf}\right)^{1.5}\sqrt{f'_{c,ksi}}$

When: $P_{no}/P_e \leq 2.25 \Rightarrow P_n = 0.658^{\frac{P_{no}}{P_e}} P_{no}$
$P_{no}/P_e \geq 2.25 \Rightarrow P_n = 0.877 P_e$

- Tensile Strength $\emptyset_c = 0.90$ $P_u \leq \emptyset_t P_n$ (neglect the tension capacity of the concrete)
  $P_n = F_y A_s + F_{y,sr} A_{sr}$

- See AISC Tables 4-13 to 4-20 for analysis/design of common filled composite columns.

Composite Steel Members – AISC Spec I

AISC Spec. I6 –Load Transfer

- AISC assumes plastic stress distribution
- Load transferred to the CONCRETE V_r' can be applied in 1 of 3 ways
  - Directly to the steel section

$$V_r' = P_r\left(1 - \frac{F_y A_s}{P_{n0}}\right)$$

  - Directly to the concrete

$$V_r' = P_r\left(\frac{F_y A_s}{P_{n0}}\right)$$

  - Applied to both steel and concrete
    - V_r' must be calculated by the difference between:
      1) The portion of the external force applied directly to the concrete and the value for AISC Eq. I6-1
      2) The portion of the external force applied directly to the steel and the value for AISC Eq. I6-2

  - Force Transfer Mechanisms
    - Direct Bearing: Limit state concrete crushing ➔ $\emptyset\, R_n = \emptyset 1.7 f_c' A_1$ where A_1 is loaded concrete area & $\emptyset = 0.65$
    - Shear Connection: Available bearing strength of shear connectors ➔ $R_n = \sum Q_{cv}$ from AISC I8.3 & Q_{cv} = sum of shear str. for connectors.
    - Direct Bond Interaction: only used with filled composite members. NOT encased members.
      - For rectangular steel section ➔ $R_n = B^2 C_{in} F_{in}$ ➔ B=overall width
      - For round steel section ➔ $R_n = 0.25\pi D^2 C_{in} F_{in}$ ➔ D=outer diameter

For both eqs: C_{in}= 2 if member extends to one side of load transf pt.
4 if member extends to two sides of ltp.
F_{in} = nominal bond stress = 0.06 ksi

Required Bond Strength ➔ $R_u \leq \emptyset R_n$ when $\emptyset = 0.45$

Steel Anchors

- Must be ≥ 1 inch of lateral clear cover
- Center-to-center spacing of stud anchors must be
  $D_{stud} \leq$ spacing $\leq 32\, D_{shank}$
- Stud diameter ≤ 2.5 $x\ base\ metal\ thickness$
- Spacing of steel channel anchors ≤ 24“

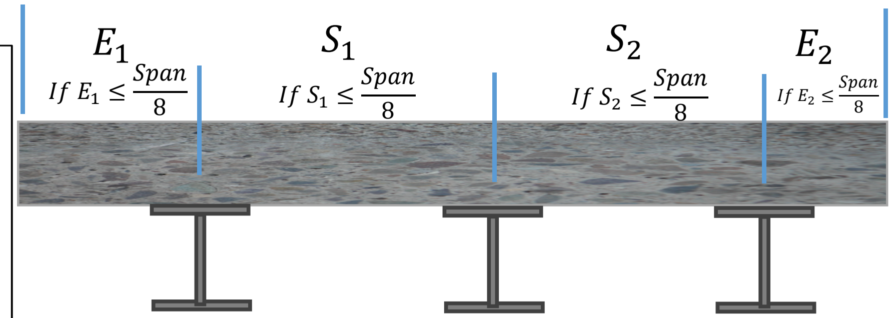
See AISC I3 for more case specific regulations

Composite Steel Members – AISC Spec I

CERM 64-5 & AISC I.1 & I.2 for Examples

<u>AISC Spec. I3 – Flexure</u>

- AISC considers
  - Encased composite beams
  - Filled composite beams
  - Steel beams w/ mechanical anchorage
    - When steel decking is used in conjunction w/ composite beams
      - The area taken up by formed steel deck can carry no compressive force
      - The direction of the deck w/ respect to composite beam matters
      - The strength of the shear studs should be adjusted to account for the deck



$b_e = E_1 + \frac{S_1}{2}$ $\quad$ $b_e = \frac{S_1}{2} + \frac{S_2}{2}$ $\quad$ $b_e = E_2 + \frac{S_2}{2}$

<u>Effective Flange Width</u>

The effective width of the concrete slab shall be the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

- $1/8^{th}$ of the beam span (measured center)
- ½ of the beam spacing (measured from beam centerline to centerline of adjacent beam)
- The distance from beam centerline to edge of slab

<u>AASHTO SPECS (not applicable to buildings)</u>

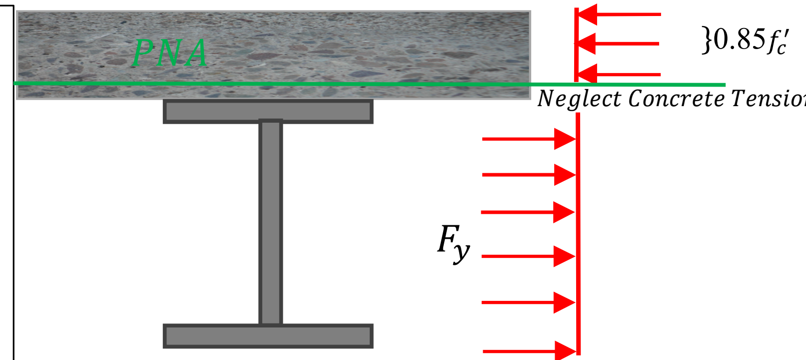
- $1/4^{th}$ beam span
- 12 times the least thickness of the slab
- Distance center to center of beams
- If slab is only on one side of the beam
  - $1/12^{th}$ of beam span
  - 6 times slab thickness
  - ½ centerlines of beam spacing

Composite Steel Members – AISC Spec I

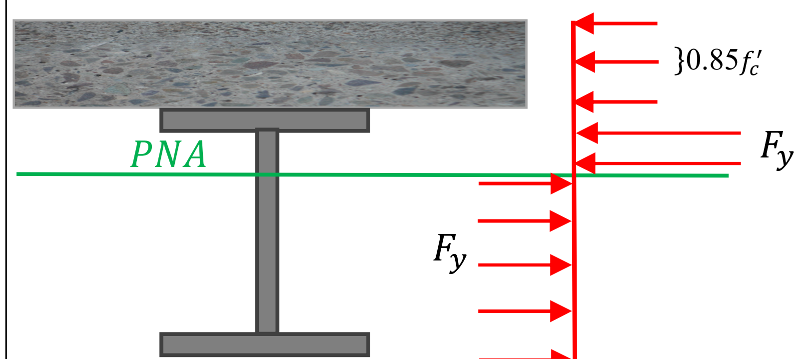
CERM 64-5 & AISC I.1 & I.2 for Examples

<u>AISC Spec. I3 – Flexure</u>

- Internal forces in the cross section
  - Location of the Plastic Neutral Axis (PNA) determines behavior
    - May be located in 1) the concrete, 2) the top steel flange, or 3) the steel web
    - Location of PNA is determined by the compressive forces in the concrete, which is the lowest of the following values (see figures to the right)
      - $A_s F_y$ (all steel in tension)
      - $0.85 f_c' A_c$ (all concrete in compression)
      - $\sum Q_n$ (maximum force that studs can transfer)
    - If $A_s F_y < 0.85 f_c' A_c$ ➔ PNA in concrete & steel controls design
    - If $A_s F_y > 0.85 f_c' A_c$ ➔ PNA in steel & concrete controls design
  - Shear Transfer: shear to the be taken by the anchors, between the points of maximum positive moment and zero moment, shall be the lowest of:
    - Concrete crushing ➔ $V' = 0.85 f_c' A_c$
    - Tensile Yield of steel section ➔ $V' = F_y A_s$
    - Strength of steel anchors ➔ $V' = \sum Q_n$
      - Steel Headed Stud Anchors ➔ $Q_n = 0.5 A_{sa}\sqrt{f_c' E_c} \le R_g R_p A_{sa} F_u$
        - A_{sa} = shank cross sectional area
        - R_g = group effect coefficient AISC I8.2a
        - R_p = position effect coefficient AISC I8.2a
        - See AISC Table 3-21 for list of Q_n values
      - Steel Channel Anchors ➔ $Q_n = 0.3(t_{fchannel} + 0.5 t_{wchannel}) L_a \sqrt{f_c' E_c}$
      - Number of Anchors ➔ # of anchors = $\frac{Horizontal\ force}{Nominal\ Str\ of\ one\ anchor\ (Q_n)}$
      - Min/Max spacing ➔ AISC I8.2c&d
      - Diameter of stud must be less than 2.5 t_f



Total horizontal force below plane btw beam & slab = $A_s F_y$



Total horizontal force above plane btw beam & slab = $0.85 f_c' A_c$

Composite Steel Members – AISC Spec I

CERM 64-5 & AISC I.1 & I.2 for Examples

AISC Spec. I3 – Flexure

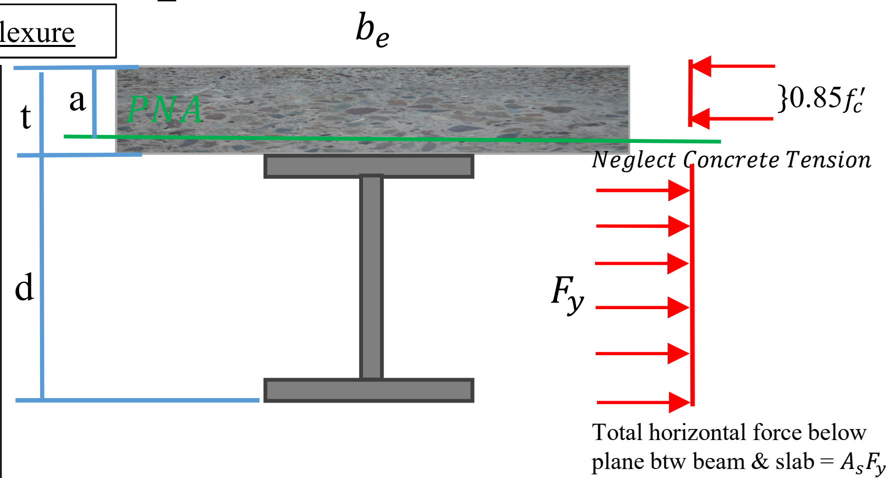
Moment Capacity $\emptyset M_n$ when **Neutral Axis in Concrete Slab**

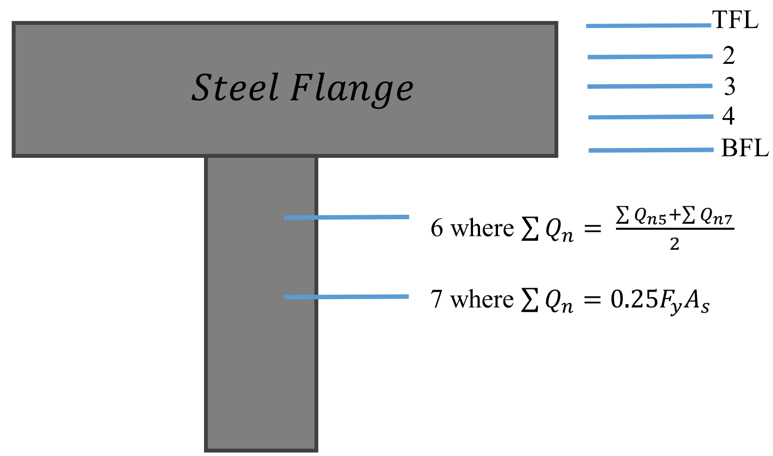
Calculation Method:

- Determine Required Flexural Strength (M_u)
- Check $\frac{h}{t_w} = \frac{(d-2k)}{t_w} < 3.76\sqrt{\frac{E}{F_{yf}}} = 90.55\ for\ 50ksi$
  - If not less than: can't use plastic stress distribution
- Approximate depth of compressive stress block
  - $a = \frac{A_s F_y}{0.85 f_c' b_e}$ ➔ $a < t$ (PNA in slab)
- $M_n = M_p = A_s F_y \left(\frac{d}{2} + t - \frac{a}{2}\right)$ ➔ $\emptyset = 0.9$
  - $\emptyset M_n > M_U$?

Express/Simplified Method:

- Determine Required Flexural Strength (M_u)
- Estimate initial moment arm for distance from top of steel beam to concrete force, Y_2. Using a=1.0 or 2.0 is ok usually. Overestimate capacity by 5% or so.
  - $Y_{2iterative} = t - \frac{a}{2}$
- AISC Table 3-19 w/ M_u & $Y_{2iterative}$ ➔ Select beam & PNA location to achieve strength
  - Try to select Y_1 appropriate for the situation (In this case $Y_1 = 0$ indicating TFL)
  - Get $\sum Q_n$ from Table 3-19
- Determine b_e
- Depth of compression block $a = \frac{\sum Q_n}{0.85 f_c' b_e}$
- Substitute into $Y_2 = t - \frac{a}{2}$ to get value of $Y_{2actual}$ for the same beam @ same location
- AISC Table 3-19 and $Y_{2actual}$ to interpolate to get actual available strength

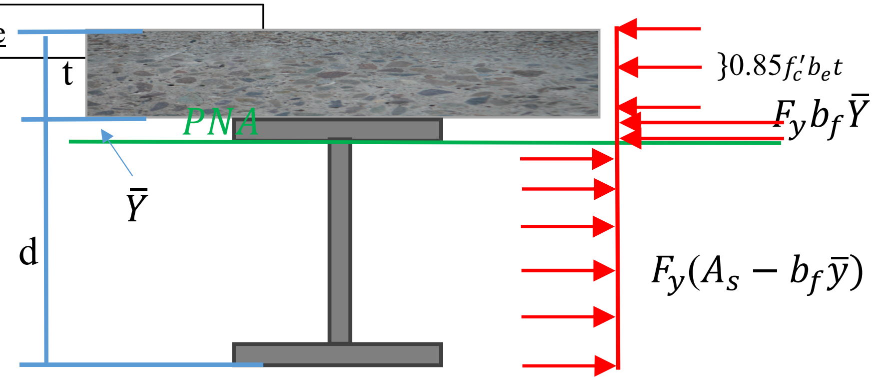




Composite Steel Members – AISC Spec I

CERM 64-5 & AISC I.1 & I.2 for Examples

AISC Spec. I3 – Flexure



Total horizontal force above plane btw beam & slab = $0.85f_c'A_c$

Calculation Method:

Moment Capacity $\emptyset M_n$ when Neutral Axis in Steel Flange/Web

- Check $\frac{h}{t_w} = \frac{(d-2k)}{t_w} < 3.76\sqrt{\frac{E}{F_{yf}}} = 90.55\ for\ 50ksi$
  - If not less than: can't use plastic stress distribution
- Approximate depth of compressive stress block
  - $a = \frac{A_sF_y}{0.85f_c'b_e}$ ➔ $a > t$ (PNA in steel)
- Is PNA in flange or web?
  - Assume base of steel flange
  - $C = 0.85f_c'b_et + F_yb_ft_f$
  - $T = F_y(A_s - b_ft_f)$
  - If C > T, PNA is in steel flange and $\bar{Y} = \frac{A_sF_y - 0.85f_c'b_et}{2b_fF_y}$
- $M_n = M_p = 0.85f_c'b_et\left(\frac{t}{2} + \bar{y}\right) + 2F_yb_f\bar{Y}\left(\frac{\bar{Y}}{2}\right) + F_yA_s\left(\frac{d}{2} - \bar{y}\right)$ ➔ $\emptyset = 0.9$
  - $\emptyset M_n > M_U$?

Express/Simplified Method:

- Determine Required Flexural Strength (M_u)
- Estimate initial moment arm for distance from top of steel beam to concrete force, Y_2. Using a=1.0 or 2.0 is ok usually. Overestimate capacity by 5% or so.
  - $Y_{2iterative} = t - \frac{a}{2}$
- AISC Table 3-19 w/ M_u & $Y_{2iterative}$ ➔ Select beam & PNA location to achieve strength
  - Try to select Y_1 appropriate for the situation (In this case $Y_1 = \bar{y}$ resulting in interpolation)
  - Get $\sum Q_n$ from Table 3-19
- Determine b_e
- Depth of compression block $a = \frac{\sum Q_n}{0.85f_c'b_e}$
- Substitute into $Y_2 = t - \frac{a}{2}$ to get value of $Y_{2actual}$ for the same beam @ same location
- AISC Table 3-19 and $Y_{2actual}$ to interpolate to get actual available strength

Composite Steel Members – AISC Spec I

AISC Spec. I3 – Flexure

- Shear Connectors
  - Strong vs weak orientation (weak is commonly chosen because engineers can't always control field installation).
    - AISC Table 3-21 ➔ Choose individual stud strength for given conditions
  - Number of Anchors per half ➔ $n_{half} = \frac{Horizontal\ force}{Nominal\ Str\ of\ one\ anchor\ (Q_n)} = \frac{\sum Q_n}{Q_{n(single\ stud)}}$
  - Full composite vs partial composite
    - When full strength of steel beam is not needed in finished structure, but may be needed during construction or to meet serviceability requirements, partial composite action is available. Cost savings

Partial Composite Method

Once a fully composite $\sum Q_n$ is established, compare it with the number of studs n_{half}

- $\sum Q_{n,pc} = n_{half} Q_n$ ➔ new concrete compression depth $a = \frac{\sum Q_{n,pc}}{0.85 f_c' b_e}$
  - Redo the section force balance➔ $C_c = 0.85 f_c' a b_e$ ➔ area of steel in comp $= A_{s,comp} = \frac{\sum Q_n - \sum Q_{n,pc}}{2F_y}$ ➔ Distance TOS to PNA $Y_1 = \frac{A_{s,comp}}{b_f}$
  - $M_n = M_p = C_c\left(t_{slab} - \frac{a}{2}\right) + A_s F_y\left(\frac{d}{2}\right) - A_{s,comp} F_y\left(\frac{Y_1}{2}\right)$

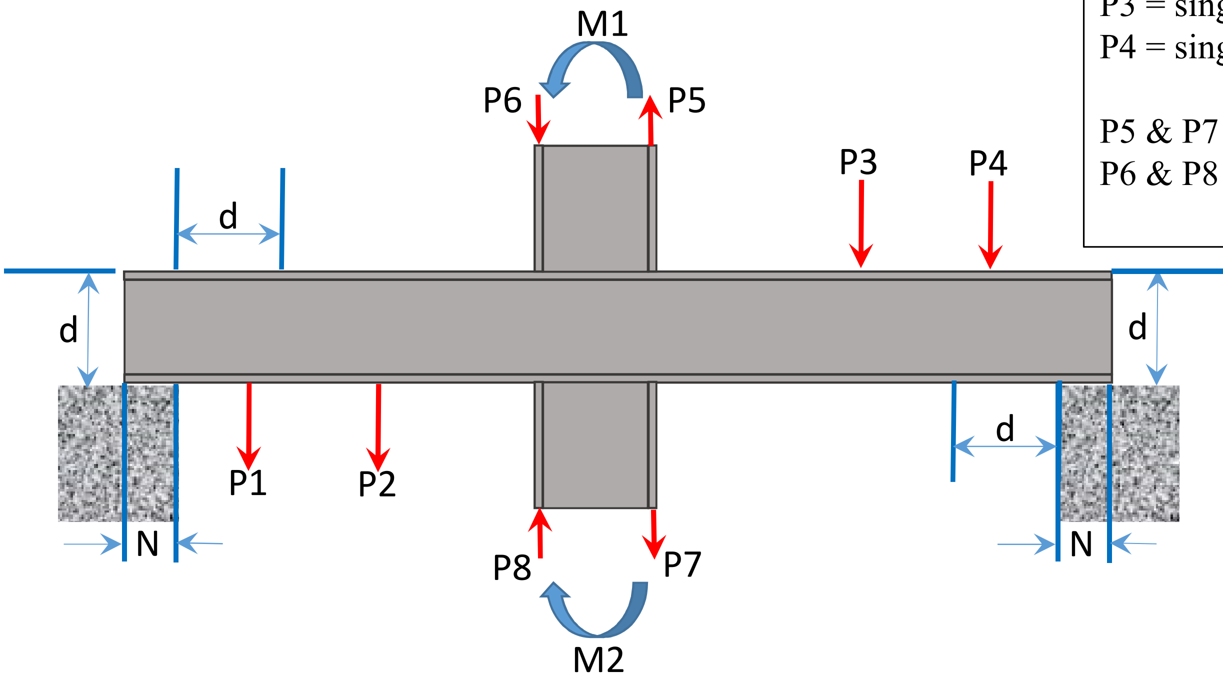
STEEL BEAM – Flange & Web Concentrated Loads

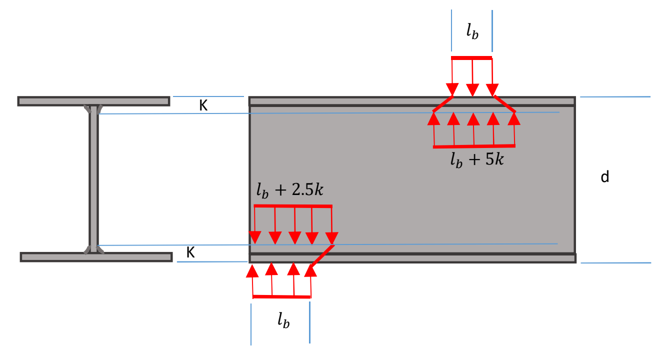
I shaped members Section J10 AISC

Nomenclature

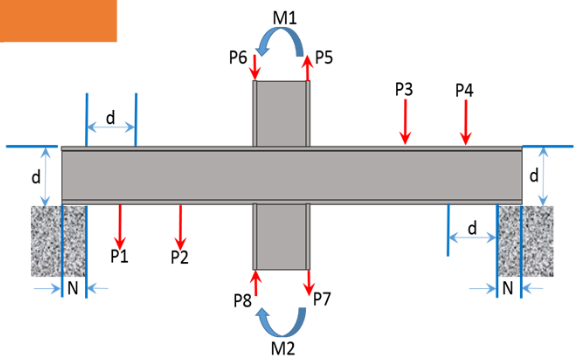
P1 = single tensile forces on flange and web within “d” of beam end
P2 = single tensile forces on flange and web outside “d” of beam end
P3 = single comp. forces on flange and web outside “d” of beam end
P4 = single comp. forces on flange and web within “d” of beam end

P5 & P7 = tensile forces doubled by M1 & M2
P6 & P8 = comp. forces doubled by M1 & M2





STEEL BEAM – Flange & Web Concentrated Loads



I shaped members Section J10 AISC

Loads within the span. NOT BEAM BEARING

What loads are present?

Load Type

Tensile Single-Concentrated Forces	Tensile Double-Concentrated Forces	Comp. Single-Concentrated Forces	Comp. Double-Concentrated Forces

Limit State to Check

Flange Local Bending $\phi = 0.9$

If load width across flange < 0.15 b_f then don't check this.

- $R_n = 6.25 F_{yf} t_f^2$ [AISC J10-1]
- If load applied ≤ $10t_f$ from end, reduce R_n by 50%.
- When required, spec transverse stiffeners.

$\phi R_n > R_u$?

Web Local Yielding $\phi = 1.00$

Load applied over "d" from beam end:

$$R_n = F_{yw} t_w (5k + l_b)$$

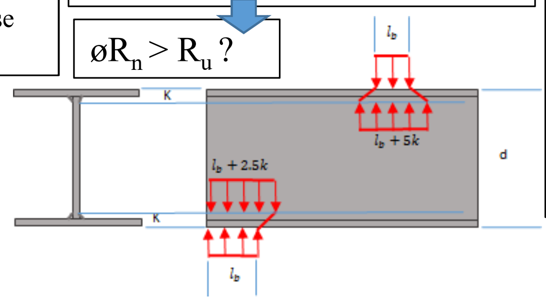
Load applied within "d" from beam end:

$$R_n = F_{yw} t_w (2.5k + l_b)$$

Use k_{des} not k_{det} for W members

When required, spec transv. stiffeners

$\phi R_n > R_u$?



Web Local Crippling $\phi = 0.75$

Load applied over d/2 from beam end:

$$R_n = 0.8 t_w^2 \left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

Load applied < d/2 from beam end:

$\frac{l_b}{d} \le 0.2$ ➔

$$R_n = 0.4 t_w^2 \left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$\frac{l_b}{d} \ge 0.2$ ➔

$$R_n = 0.4 t_w^2 \left(1 + \left(\frac{4 l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$\phi R_n > R_u$?

Web Sideways Crippling (See next page)

Web Compression Buckling (See next page)

Web Panel Zone Shear (See next page)

Specification Section J10 Limit States

Section J10 of the 2005 AISC *Specification* specifies several parameters to resist local failures. However, after reading descriptions before each limit state, I still find it difficult to determine which limit states apply to a given load. For instance, it is hard to picture what "a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location" looks like. I would be grateful if you could please provide clarification of the descriptions for the limit states in Section J10.

First you should keep in mind that some of the limit states apply only to tension loads, some to only compressive loads, and some to both. This is discussed in the *Specification* Commentary on page 16.1-355 as follows:

Single concentrated forces may be tensile (such as those delivered by tension hangers) or compressive (such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections). Flange local bending applies only for tensile forces, web local yielding applies to both tensile and compressive forces, and the remainder of these limit states apply only to compressive forces. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections.

Flange local bending applies only to tension forces, so it need not be checked where only compression will occur such as in seated connections. It applies to both single forces, such as hangers, and double forces, such as moment connections.

Web local yielding applies to both tension and compression forces, and also applies to both single and double forces. It would be checked, for example, at seats, hangers, and moment connections. It is also usually checked at the gusset-to-beam interface of vertical bracing connections.

Web crippling applies only to compression forces, but applies to both single and double forces. It would be checked, for example, at seats and moment connections, but not hangers. It is also usually checked at the gusset-to-beam interface with vertical bracing connections, if compression can exist.

Web sidesway buckling applies only to compression forces, and applies only to single forces such as a column bearing on an unrestrained beam.

Web compression buckling, like web crippling, applies only to compression forces, and applies to both single and double forces. However the compression force must be applied to both sides of the member, so that the web acts similar to a column. This limit state is not checked, for instance, with seats or where a moment connection is present on only one side of the column.

Web panel zone shear only needs to be checked where double forces lead to a shear in the web.

Larry S. Muir, P.E.

STEEL BEAM – Beam End Bearing Requirements AISC Ch 9. PPI study book pg. 6-10

AISC Manual Table 9-4 uses $l_b = 3\frac{1}{4}$": Beam End Bearing Constants

$R_1 = 2.5kF_{yw}t_w$

$R_2 = F_{yw}t_w$

$R_3 = 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}$

$R_4 = 0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$

$R_5 = 0.40t_w^2\left(1 - 0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$

$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$

Web Local Yield @ Beam Ends

Comp. Force Applied < "d" from beam end:

$$\emptyset R_n = \emptyset R_1 + l_b(\emptyset R_2)$$

$$l_b = \frac{\emptyset R_n - \emptyset R_1}{\emptyset R_2} > k$$

Comp. Force Applied ≥ "d" from beam end:

$$\emptyset R_n = 2\emptyset R_1 + l_b(\emptyset R_2)$$

$$l_b = \frac{\emptyset R_n - 2\emptyset R_1}{\emptyset R_2} > k$$

Web Local Crippling @ Beam Ends

Comp. Force Applied < "d/2" from beam end:

If $\frac{l_b}{d} \leq 0.2$

$$\emptyset R_n = \emptyset R_3 + l_b(\emptyset R_4)$$

$$l_b = \frac{\emptyset R_n - \emptyset R_3}{\emptyset R_4} > k$$

$\frac{l_b}{d} > 0.2$

$$\emptyset R_n = \emptyset R_5 + l_b(\emptyset R_6)$$

$$l_b = \frac{\emptyset R_n - \emptyset R_5}{\emptyset R_6} > k$$

Comp. Force Applied ≥ "d/2" from beam end:

$$\emptyset R_n = 2\left(\emptyset R_3 + l_b(\emptyset R_4)\right)$$

$$l_b = \frac{\emptyset R_n - 2\emptyset R_3}{2\emptyset R_4} > k$$

STEEL BEAM – Bearing Plate Design & Bearing on Concrete & Masonry

Bearing on Concrete:
Section 10.14 of ACI 318 &AISC Spec J8
- Loaded Area 1v:2h slope
- Design strength increase based on $\sqrt{A_2/A_1}$ proportionality

See pg 6-13 & design example in Steel design PE study guide.
- Bearing plate thickness
- Plastic section modulus
- AISC Part 14
- Also See page 334 Mccormack Steel textbook

Bearing on Masonry:
Section 1.9.5 of ACI 530

STEEL BEAM – Stiffener & Double Plate Requirements

Check R_u against
- ϕR_n for Web Local Yielding
- ϕR_n Web Crippling

If $\phi R_n < R_u$ Use a stiffener!

How much strength does the stiffener need to handle? (It's the difference between the applied load and what the unstiffened beam can handle)

$$R_{st} = R_u - \emptyset R_n$$

Determine min/max stiffener widths

$b_{st,max} = \frac{b_f - t_w}{2}$ $\quad b_{st,min} = \frac{b_f - t_w}{6}$

Choose width closer to the max

Determine min stiffener thickness

$$h_{st,min} = d - 2k_{des}$$

Check Coeff. of slender unstiffened elements in compression (k_c):

AISC Table B4.1A (Note a)

$$k_c = \frac{4}{\sqrt{\frac{h_{st}}{t_w}}} \qquad [0.35 \; k_c \leq \; 0.76]$$

Calculate limiting width/thickness ratio for stiffener:

$$\frac{b_{st}}{t_{st}} \leq 0.64\sqrt{\frac{k_c E}{F_y}} \rightarrow t_{st,min} \geq \frac{b_{st}}{0.64\sqrt{\frac{k_c E}{F_y}}}$$

Verify Stiffeners Meet Required Strength:
- Determine Gross Area of Cross Shaped Column
- $A_{g,cross} = A_{st} + 12t_w^2$ if end stiffener
  $A_{st} + 25t_w^2$ if interior stiffener
  when: $A_{st} = n_{st} b_{st} t_{st}$
- Effective web length = $L_{w,eff} = 12t_w$ if end stiffener
  $= 25t_w$ if interior stiffener
- Calculate Moment of Inertia of the Cross-shaped Column (I_{cross})

$$I_{cross} = I_{st} + I_w = \frac{(bd^3)_{st}}{12} + \frac{(bd^3)_w}{12} = \frac{t_{st}(t_w + 2b_{st})^3}{12} + \frac{(L_{w,eff} - t_{st})t_w^3}{12}$$

- Calculate Effective Slenderness Ratio (Eff. Length Factor = 0.75)

$$\frac{KL}{r} = \frac{Kh_{cross}}{r_{cross}} \qquad \text{when: } r_{cross} = \sqrt{\frac{I_{cross}}{A_{g,cross}}}$$

Nominal Axial Compression Load Capacity $\quad P_n = F_{cr} A_{g,cross}$

When Effective slenderness ratio

$$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = 0.658^{\left(\frac{F_y}{F_e}\right)} F_y$$

When Effective slenderness ratio

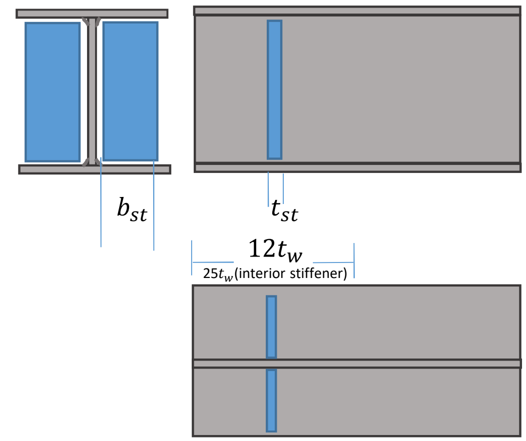
$$\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$$

$$F_{cr} = 0.877 F_e$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Calculate the Design Strength: $P_{u,st} = \emptyset_c P_n$

$P_{u,st} > R_{st}$?



Connecting Elements – AISC Ch. 9

AISC Chapter 9: Limit States

<u>AISC Part 9</u>: Design of Connecting Elements

- Gross area (A_g) and effective net area (A_n) may have effective width modified by "**Whitmore Section**"
  - When connecting elements are large compared to bolted/welded joints, the Whitmore Section limits the gross and net areas of connecting elements to less than the full area.
    - l_w is determined by spreading the force at the start of the joint by 30^o along the line of force.

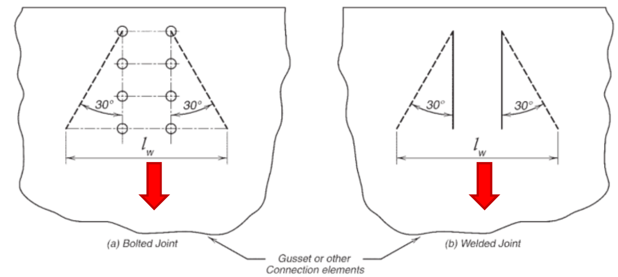


Fig. 9-1. Illustration of the width of the Whitmore section.

- Connecting elements subject to **Tension**
  - Available strength due to tension yielding & rupture, $\emptyset R_n$ must exceed R_u per AISC Spec. J4.1
    - Tensile yielding: $R_n = F_y A_g$ w/ $\emptyset = 0.9$
    - Tensile rupture: $R_n = F_u A_e$ w/ $\emptyset = 0.75$
      - A_e is the effective net area from section D3. For bolted splice plates, $A_e = A_n \leq 0.85 A_g$

- Connecting elements subject to **Block Shear Rupture**
  - Available strength due to block shear rupture, $\emptyset R_n$ must exceed R_u per AISC Spec. J4.3. AISC Table 9-3 is used to calculate BSR.

- Connecting element **Rupture Strength at Welds**
  - Check used to calculate min. base metal thickness that matches available shear rupture strength of base metal to the shear rupture strength of welds.
    - Fillets on 1 side of conn. element, $t_{\min(element)} = \frac{0.6F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_{u(element)}} = \frac{3.09D}{F_{u(element)}}$ Weld on both sides ➔ $t_{\min(element)} = \frac{6.19D}{F_{u(element)}}$

- Connection elements subject to **Compression Yielding & Buckling**
  - Available strength $\emptyset P_n$ must exceed P_u per AISC Spec. J4.4.
    - When $\frac{KL}{r} \leq 25$ ➔ $P_n = F_y A_g$ w/ $\emptyset = 0.9$
    - When $\frac{KL}{r} > 25$ ➔ Use Chapter E (Part E3 most likely)
      - $P_n = F_{cr} A_g$ w/ $\emptyset = 0.9$
        - When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = \left[0.658^{\left(\frac{F_y}{F_e}\right)}\right] F_y$ Where: $F_e(elastic\ buckling\ stress) = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$
        - When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ ➔ $F_{cr} = 0.877 F_e$

Connecting Elements

AISC Chapter 9: Limit States

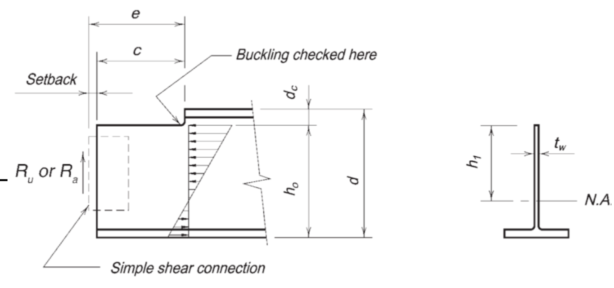


Fig. 9-2. Flexural local buckling of beam web coped at top flange only.

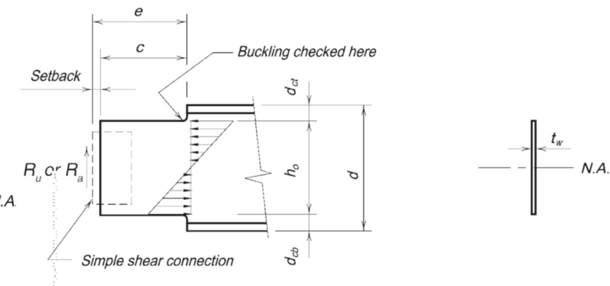


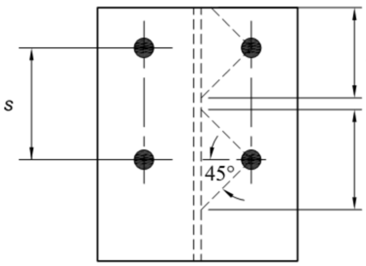
Fig. 9-3. Flexural local buckling of beam web coped at both flanges.

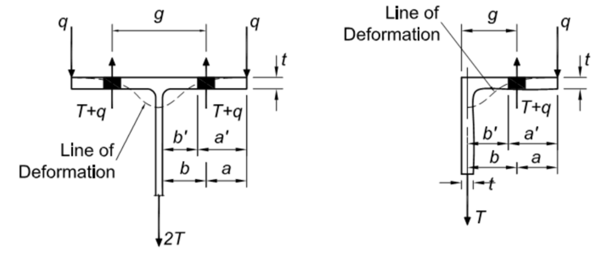
AISC Part 9: Design of Connecting Elements

- Affected Elements subjected to **Flexure**
  - Must satisfy the following, in accordance w/ AISC Spec./Table F1.1
    - Flexural yielding, Lateral-Torsional Buckling, & Local Buckling
    - Rupture: For beams/girders w/ bolt holes in tension flange ➔
      - AISC spec. F13.1 (also pg 5-38 in PPI book)
      - For other connecting elements, available flexural rupture strength ➔ $M_n = F_u Z_{net}$ w/ $\emptyset = 0.75$
  - Strength of **Coped Beams**
  - For a coped beam, the **required flexural strength** $M_u = R_u e$ Where:
      - R_u is the beam end reaction
      - "e" = distance from face of cope to the point of inflection of the beam (inches). The point of inflection may conservatively be taken as the distance from the cope face to the surface of the supporting member (shown in figure).
    - The **Available Flexural Local Buckling Strength** of a beam w/ coped flanges must exceed M_u
      - $M_n = F_{cr} S_{net}$ w/ $\emptyset = 0.9$ where F_{cr} & S_{net} depend upon the extent to which each flange is coped.
        - **Coped @ Top Flange Only**: $F_{cr} = \frac{\pi^2 E}{12(1-v^2)}\left(\frac{t_w}{h_o}\right)^2 fk \le f_y$ ➔ commonly for E=29000ksi ➔ $F_{cr} = 26{,}210\left(\frac{t_w}{h_o}\right)^2 fk \le f_y$
          - Where: E=29,000 ksi, $v = 0.3$, $F_y = beam\ web\ mtrl\ yield$, $f = \begin{cases} \frac{2c}{d} & when\ \frac{c}{d} \le 1.0 \\ 1 + \frac{c}{d} & when\ \frac{c}{d} > 1.0 \end{cases}$ & $k = \begin{cases} 2.2\left(\frac{h_o}{c}\right)^{1.65} & when\ \frac{c}{h_o} \le 1.0 \\ 2.2\frac{h_o}{c} & when\ \frac{c}{h_o} > 1.0 \end{cases}$
        - **Same Cope Length @ Both Flanges** when $c \le 2d$ & $d_c \le 0.2d$: $F_{cr} = 0.62\pi E\frac{t_w{}^2}{ch_o} f_d \le F_y$ where: $f_d = 3.5 - 7.5\left(\frac{d_{ct}}{d}\right)$
        - **All Other Cases**: $F_{cr} = QF_y$ with $\lambda = \frac{h_o\sqrt{F_y}}{10t_w\sqrt{475+280\left(\frac{h_o}{c}\right)^2}}$, $Q = \begin{cases} 1 & when\ \lambda \le 0.7 \\ 1.34 - 0.486\lambda & when\ 0.7 < \lambda \le 1.41 \\ \frac{1.30}{\lambda^2} & when\ \lambda > 1.41 \end{cases}$
        - When the **Tension Flange Cope Longer than Comp. Flange Cope**: Check Flexural Yielding @ end of tension flange cope ➔ $M_n = F_y S_{net(end)}$ w/ $\emptyset = 0.9$

Connecting Elements

AISC Chapter 9:
Prying, Rotational Ductility, & Shims/Fillers



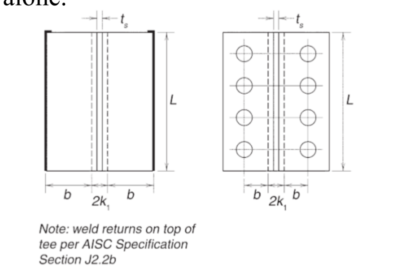


(a) Prying forces in tee (b) Prying forces in angle

Fig. 9-4. Illustration of variables in prying action calculations.

<u>AISC Part 9</u>: Design of Connecting Elements

- **Prying Action** is when deformation of a connecting element under a tensile force increases the tensile force in the bolts due to applied tensile force alone.
  - Design includes sizing bolts & connection element thicknesses so there is sufficient strength. Based on F_u
  - Example on pg 453 McCormack Textbook & AISC Design Example II.D-1
  - The **Thickness Required to Eliminate Prying Action** is:
    - $t_{min} = \sqrt{\frac{4Tb'}{\emptyset p F_u}}$ (This essentially ensures that "q" is equal to 0).
      - Where: F_u =min. Tensile strength of element, T=required strenth per bolt (r_{ut}), b'=$b - \frac{d_b}{2}$,
      - b=dist. from bolt centerline to center leg of angle or face of "T", & $p = tributary\ bolt\ length \Rightarrow max = 2b, but\ \le s$
  - Other cases, including some contribution of "q", are included on pg 9-12 of AISC Steel Manual



(a) Welded flange (b) Bolted flange

Fig. 9-5. Illustration of variables in shear connection ductility checks.

- **Rotational Ductility** is required for simple connections by AISC Spec. J1.2 for the following:
  - Dbl. Angle, Shear End-Plate, Single-Angle, & T-Shear Connections ➔ geometry & thickness of conn. element to support are configured to allow flexure in simple-beam end rotation.
  - Stiffened & Unstiffened seated Connections ➔ geometry & thickness of top/side stability angle is configured to allow connecting element to accommodate simple-beam end rotation.
  - Single-Plate connections ➔ geometry & thickness of plate configured so the plate will yield, bolt group will rotate, and/or bolt holes will elongate **prior** to failure of welds or bolts.
  - AISC Part 10 has guidance for the above cases, except T-shear (which can be found on AISC pg 9-14).

- **Shims & Fillers** are used in simple-shear connections, PR, and FR moment connections, column base-plates, and column splices.
  - Strip shims are cheaper to make, but finger shims can be inserted laterally and don't require erection bolts to be removed
  - Finger shims, inserted fully against the bolt shank, are acceptable in slip-critical connections and are not to be considered an internal ply w/ SSL holes
    - Because less than 25% of available contact surface is lost, which is not enough to affect joint performance.
  - AISC J3.8 (SC bolted connections) & J5.2 (fillers in bolted connections) describe effect of fillers and shims on joint strength.
- **Web Reinforcement of Coped Beams**
  - See AISC pg 9-17 for stipulations

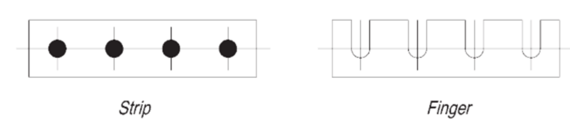


Fig. 9-6. Shims.

Simple Shear Connections

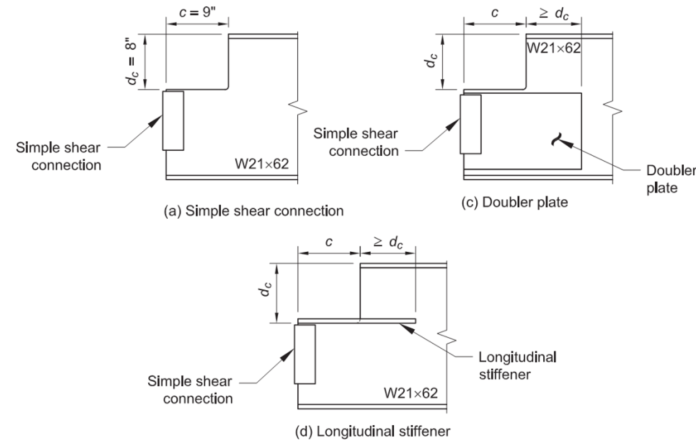
AISC Chapter 9:

AISC Part 9: Design of Connecting Elements

- **Prying   pg 453 Mccormakck textbook.**

Simple Shear Connections

Beam End Coped At Top Flange



Doubler Plates @ Simple Shear Connections (AISC Design Example II.A-6)

- When a given design reaction exceeds the available strength of a coped beam, a doubler plate may be used to compensate
  - Double plate required strength ➔ $R_u - \emptyset R_{n(beam\ only)} = Additional\ Strength\ Needed$
  - Additional Section Modulus Required to satisfy additional strength ➔ $S_{req} = \frac{(R_u - \emptyset R_{n(beam\ only)})e}{\emptyset F_y}$
    - e should include ½" setback.
  - Proper doubler plate thickness ➔ $t_{req} = \frac{6S_{req}}{(doubler\ plate\ depth)^2}$
  - Use plate material to match beam material's strength. ASTM A572-50 works with A992 beams.
  - Doubler plate must extend at least the vertical depth of the cope past the end of the horizontal cope.
  - Size plate accordingly and use fillet welds on top and bottom to attach to beam web.

Longitudinal Stiffener Design

Try PL¼ in.×4 in. slotted to fit over the beam web with F_y = 50 ksi.

From section property calculations for the neutral axis and moment of inertia, conservatively ignoring the beam fillets, the neutral axis is located 4.39 in. from the bottom flange (8.86 in. from the top of the stiffener).

	I_o (in.^{4})	Ad^2 (in.^{4})	$I_o + Ad^2$ (in.^{4})
Stiffener	0.00521	76.3	76.3
W21×62 web	63.3	28.9	92.2
W21×62 bottom flange	0.160	84.5	84.7
			$\Sigma = I_x = 253$ in.^{4}

Longitudinal Stiffener @ Simple Shear Connections (AISC Design Example II.A-6)

- Isolate the N.A. with a proposed Stiffener N.A. Height ➔ $\frac{\sum A\bar{y}}{\sum A}$ ➔ 4.39" in the example to the right.
- Evaluate Slenderness of Long. Stiffener per AISC Spec. B4.1b Case 11
  - $\lambda_r = 0.95\sqrt{\frac{K_c E}{F_L}}$ Where: $0.35 \le K_c = \frac{4}{\sqrt{\frac{h}{t_w}}} \le 0.76$
  - Calculate the ratio of the section moduli: $S_{xc} = \frac{I_x}{c(comp)}$ $v = \frac{I_x}{c(tension)}$ Compression is on top of the beam
  - $\frac{S_{xt}}{S_{xc}}$ ➔ Table B4.1b footnote [b] to determine which F_L value. ➔ calculate λ_r
  - $\frac{b_{long.stiffener}}{t}$ (using b as ½ bflange) ➔determine if stiffener is slender. Upsize it if it is slender.
  - $S_{net} = S_{xc}$ ➔ Nominal strength of reinforced section ➔ $\boldsymbol{R_n} = \frac{\boldsymbol{F_y S_{net}}}{\boldsymbol{e}}$ w/ $\emptyset = \mathbf{0.9}$ & $\boldsymbol{\Omega} = \mathbf{1.67}$
  - Long. Stiffener must extend at least the vertical depth of the cope past the end of the horizontal cope
  - Size plate accordingly and use fillet welds on top and bottom to attach to beam web.

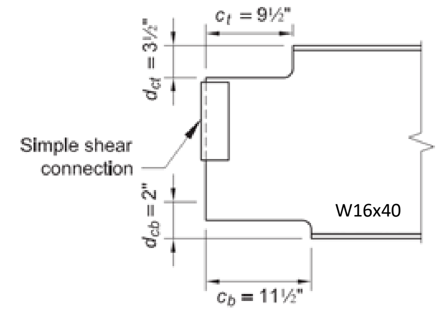
Simple Shear Connections

Beam End Coped At Top & Bottom Flanges

Doubler Plates @ Simple Shear Connections (AISC Design Example II.A-7)

- When top and bottom are coped, use the equations on pg 9-9 of AISC to determine F_{cr}
  - This is also on the previous slide for coped beams.
  - Local Buckling of Compression (Top) Flange Cope
    - $S_{net} = \frac{t_w {h_o}^2}{6}$
    - $M_n = F_{cr} S_{net}$
    - $R_n = \frac{M_n}{e_{top}}$ ➔ w/ $\emptyset = \mathbf{0.9}$ & $\boldsymbol{\Omega} = \mathbf{1.67}$
  - Flexural Yielding of Tension (Bottom) Flange Cope
    - Ignoring the 3.5" compression cope ➔ Use Table 9-2 to get $S_{net} = 15.6\ in^3$
    - $M_n = F_y S_{net}$
    - $R_n = \frac{M_n}{e_{bottom}}$ ➔ w/ $\emptyset = \mathbf{0.9}$ & $\boldsymbol{\Omega} = \mathbf{1.67}$
  - **Lower of the two values governs!**

For an ASTM A992 W16×40 coped 3½ in. deep by 9½ in. wide at the top flange and 2 in. deep by 11½ in. wide at the bottom flange calculate the available strength of the beam end, considering the limit states of flexural yielding and local buckling. Assume a ½-in. setback from the face of the support to the end of the beam.



From AISC *Manual* Table 1-1 and AISC *Manual* Figure 9-3, the geometric properties are as follows:

d = 16.0 in.
t_w = 0.305 in.
t_f = 0.505 in.
b_f = 7.00 in.
c_t = 9.50 in.
d_{ct} = 3.50 in.
c_b = 11.5 in.
d_{cb} = 2.00 in.
e_b = 11.5 in. + 0.50 in.
= 12.0 in.
e_t = 9.50 in. + 0.50 in.
= 10.0 in.
h_o = 16.0 in. − 2.00 in. − 3.50 in.
= 10.5 in.

Simple Shear Connections

AISC Chapter 10:

Simple Shear Connections

- The ends of simple shear connections assumed to be free to rotate under loads
- Comparing Connection Alternatives
  - **Two-Sided Connections** – double angles, shear end plates
    - Good when end reaction is large
    - Compact ➔ contained within flanges
    - Helps regulate/minimize eccentricities
  - **Seated Connections** – Stiffened or unstiffened
    - Shop attached option
    - Ample erection clearances
    - Safe erection
  - **One-Sided Connections –** Single-plate, Single-angle, tee connections
    - Shop attachment to support
    - Reduced material and shop labor
    - Excellent safety because no double connections

Design Tables

- Table 10-1 ➔ All bolted Double-Angle Connections
  - Supported & Supporting members $F_y = 50ksi$ & $F_u = 65ksi$
  - Angle Material $F_y = 36ksi$ & $F_u = 58ksi$
  - Eccentricity neglected for distance btw face of angles to CL of bolts ≤ 3 *inches*
- Steps to use the table:
  1. Establish member sizes and support thicknesses
  2. Required strength/design loads
  3. Select bolt rows and trial angle size based on geometry constraints
  4. Evaluate a) Strength of connecting elements, b) beam we strength, and c) available strength of support.

- Strength of Connecting Elements:
  - Bolt shear
  - Bolt bearing on angles
  - Shear yielding of angles
  - Shear rupture of angles
  - Block shear of angles

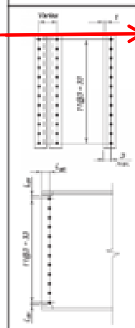
- Beam web strength per inch of thickness:
  - Depends on the cope: L_{ev}

- Available strength of Support per inch of thickness

Beam: $F_y = 50$ ksi, $F_u = 65$ ksi
Angle: $F_y = 36$ ksi, $F_u = 58$ ksi

Table 10-1
All-Bolted Double-Angle Connections
3/4-in. Bolts

12 Rows
W44



Bolt and Angle Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD	197	295	246	369	286	430	286	430
	X	STD	197	295	246	369	295	443	361	541
	SC Class A	STD	152	228	152	228	152	228	152	228
		OVS	129	194	129	194	129	194	129	194
		SSLT	152	228	152	228	152	228	152	228
	SC Class B	STD	197	295	246	369	253	380	253	380
		OVS	196	294	216	323	216	323	216	323
		SSLT	195	293	244	366	253	380	253	380
Group B	N	STD	197	295	246	369	295	443	361	541
	X	STD	197	295	246	369	295	443	393	590
	SC Class A	STD	190	285	190	285	190	285	190	285
		OVS	162	242	162	242	162	242	162	242
		SSLT	190	285	190	285	190	285	190	285
	SC Class B	STD	197	295	246	369	295	443	316	475
		OVS	196	294	245	367	270	403	270	403
		SSLT	195	293	244	366	293	440	316	475

(Angle Thickness, in.)

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type / L_{ev}, in.		STD L_{eh}* 1 1/2 ASD	STD 1 1/2 LRFD	STD 1 3/4 ASD	STD 1 3/4 LRFD	OVS 1 1/2 ASD	OVS 1 1/2 LRFD	OVS 1 3/4 ASD	OVS 1 3/4 LRFD	SSLT 1 1/2 ASD	SSLT 1 1/2 LRFD	SSLT 1 3/4 ASD	SSLT 1 3/4 LRFD
Coped at Top Flange Only	1 1/4	498	747	506	759	468	702	476	714	495	743	503	755
	1 3/8	501	751	509	763	470	706	479	718	497	746	506	758
	1 1/2	503	754	511	767	473	709	481	722	500	750	508	762
	1 5/8	505	758	514	770	475	713	483	725	502	753	510	766
	2	513	769	521	781	483	724	491	736	510	764	518	777
	3	532	798	540	810	502	753	510	765	529	794	537	806
Coped at Both Flanges	1 1/4	488	731	488	731	458	687	458	687	488	731	488	731
	1 3/8	492	739	492	739	463	695	463	695	492	739	492	739
	1 1/2	497	746	497	746	468	702	468	702	497	746	497	746
	1 5/8	502	753	502	753	473	709	473	709	502	753	502	753
	2	513	769	517	775	483	724	488	731	510	764	517	775
	3	532	798	540	810	502	753	510	765	529	794	537	806
Uncoped		702	1050	702	1050	702	1050	702	1050	702	1050	702	1050

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1400	2110

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance, L_{eh}, to account for possible underrun in beam length.

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

Simple Shear Connections

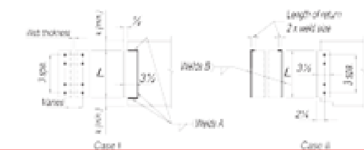
Double Angle Connections

Bolted/Welded Double Angle Connection

- Table 10-2 ➔ When double angle connection includes either 1) weld to beam web or 2) weld to support element.
- Steps to use the table:
    1. Use it in conjunction with Table 10-1, but substitute for appropriate check.
    2. Be sure to check minimum support and web thicknesses
    3. Minimum Angle Thickness AISC J2.2b based on fillet weld size.

- Beware of Case I vs Case II

Table 10-2
Available Weld Strength of Bolted/Welded Double-Angle Connections



n	L, in.	Welds A (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Welds B (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
12	35 1/2	5/16	393	589	0.476	3/8	366	550	0.286
		1/4	314	471	0.381	5/16	305	458	0.238
		3/16	236	353	0.286	1/4	244	366	0.190
11	32 1/2	5/16	365	548	0.476	3/8	331	496	0.286
		1/4	292	438	0.381	5/16	276	414	0.238
		3/16	219	329	0.286	1/4	221	331	0.190

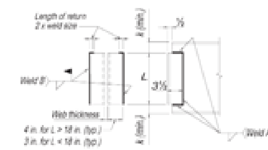
Fully Welded Double Angle Connection

- Table 10-3 ➔ When double angle connection includes both 1) weld to beam web and 2) weld to support
- Steps to use the table:
    1. Replaces Table 10-1.
    2. Be sure to check minimum support & web thickness.
    3. Check shear yielding of angles along weld length.

$$A_{gv} = 2(Length_{weld})(Thickness_{angle})$$

$$R_n = 0.6F_yA_{gv} \rightarrow \emptyset = 1.0 \;\&\; \Omega = 1.5$$

Table 10-3
Available Weld Strength of All-Welded Double-Angle Connections



L, in.	Welds A (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Welds B (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.
36	5/16	397	596	0.476	3/8	372	558	0.286
	1/4	318	477	0.381	5/16	310	465	0.238
	3/16	238	357	0.286	1/4	248	372	0.190
34	5/16	379	568	0.476	3/8	349	523	0.286
	1/4	303	455	0.381	5/16	291	436	0.238
	3/16	227	341	0.286	1/4	232	349	0.190

Simple Shear Connections

Double Angle Connections

Note: The middle portion of AISC *Manual* Table 10-1 includes checks of the limit-state of bolt bearing on the beam web and the limit-state of block shear rupture on coped beams. AISC *Manual* Tables 9-3a, 9-3b and 9-3c may be used to determine the available block shear strength for values of L_{ev} and L_{eh} beyond the limits of AISC *Manual* Table 10-1. For coped members, the limit states of flexural yielding and local buckling must be checked independently per AISC *Manual* Part 9.

Double Angle Connections w/ Beam Copes (AISC Design Example 11.A-4)

- Use appropriate Table 10-# for the Weld/Bolt Orientation
    - Incorporate L_{ev} to account for weakened beam section
    - Block Shear strength from Tables 9-3a,b,c may be used for L_{ev} & L_{eh} over those given in Table 10-1.
        - Table 10-1 values include checks on limit-state of bolt bearing and block shear rupture.
        - So limit states of 1) Flexural Yielding & 2) Local Buckling must be checked independently.
- Flexural Local Web Buckling
    - See previous slides on AISC Ch. 9
        - Depending on the relationship of "c" & "d", isolate F_{cr}
        - $R_n = \frac{F_{cr} S_{net}}{e}$ ➔ $\emptyset = 0.9$ & $\Omega = 1.67$
- Shear Yielding of Beam Web ➔ $R_n = 0.6 F_y A_{gv}$ ➔ $R_n = 0.6 F_y t_w h_o$ ➔ $\emptyset = 1.0$ & $\Omega = 1.5$
- Shear Rupture of Beam Web ➔ $R_n = 0.6 F_u A_{nv}$ ➔ $R_n = 0.6 F_u (t_w)[h_o - n(d_{bolt} + 0.125)]$ ➔ $\emptyset = 0.75$ & $\Omega = 2.0$

Simple Shear Connections

Single Plate Connections – Shear Tabs   AISC 10-102

Conventional Configuration
- Dimensional Limitations
  - One vertical column of bolts (2 to 12 bolts)
  - Dimension “a” must be ≤ 3.5”
  - Std. or short slotted holes as noted in Table 10-9
  - Vertical edge distance L_{ev} must satisfy Table J3.4
  - Horizontal edge distance $L_{eh} \geq 2d_b$
  - Plate thickness or beam web thickness must satisfy max in Table 10-9.
- Design Checks
  - Bolts & Plate must be checked for shear with eccentricity “e” give in Table 10-9.
  - Plate buckling typically doesn’t control for conventional configuration.

Extended Configuration
- Procedure for extended config. and conventional w/ multiple bolt columns.
- Dimensional Limitations
  - # of bolts, n, not limited
  - Distance from weld line to bolt line, a, not limited
  - Holes in accordance w/ J3.2
  - L_{ev} & L_{eh} to satisfy Table J3.4
- Design Checks
  - Bolt group required for given eccentricity
  - Max plate thickness so that plate moment strength doesn’t exceed the moment strength of the bolt group in shear
  - Plate: Shear yield, shear rupture, block shear
  - Plate: shear yield, shear buckling, yielding due to flexure
  - Dbl coped beam procedure Ch 9.
  - Support beam bracing

AISC Table 10-10
- Evaluated for “a”=3”, but valid for “a” btw 2.5 & 3 inches.

- Accounts for:
  - Bolt Shear,
  - Bolt Bearing,
  - Shear Yielding,
  - Shear Rupture,
  - Block Shear of End-Plate

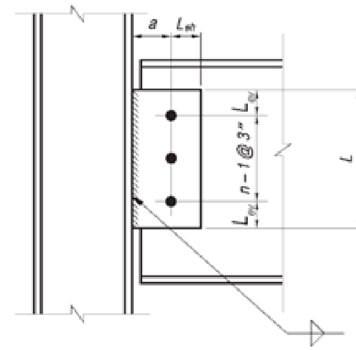


Fig. 10-11. Single-plate connection.

Table 10-9
Design Values for Conventional Single-Plate Shear Connections

n	Hole Type	e, in.	Maximum t_p or t_w, in.
2 to 5	SSLT	a/2	None
	STD	a/2	d/2 + 1/16
6 to 12	SSLT	a/2	d/2 + 1/16
	STD	a	d/2 − 1/16

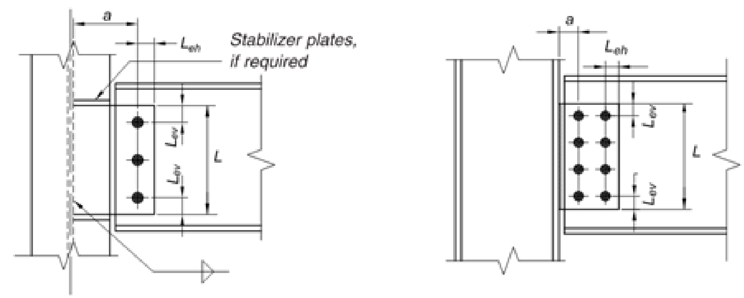


Fig. 10-12. Single-plate connection—Extended Configuration.