REFERENCES	CALCULATIONS	RESULTS
	Beam to Column Web Shear Connection Calculations **Double Angle Connection | AISC 360-16 LRFD**	
	Design Load/s: $V_u = 30\ \mathrm{kip}$ - Vertical Shear Load	
	Beam Section Properties: $\mathrm{W18x35}$ - Beam Size $d_{bm} = 17.7\ \mathrm{in}$ - Beam Depth $t_{wb} = 0.3\ \mathrm{in}$ - Beam Web Thickness $b_{fb} = 6\ \mathrm{in}$ - Beam Flange Width $t_{fb} = 0.425\ \mathrm{in}$ - Beam Flange Thickness $A_{bm} = 10.3\ \mathrm{in}^2$ - Beam Area **Beam Grade Information:** $\mathrm{A992}$ - Material Grade $F_{yb} = 50\ \mathrm{ksi}$ - Beam Yield Stress $F_{ub} = 65\ \mathrm{ksi}$ - Beam Tensile Stress $E = 29000\ \mathrm{ksi}$ - Beam Modulus of Elasticity	
	Column Section Properties: $\mathrm{W14x90}$ - Column Size $d_{sup} = 14\ \mathrm{in}$ - Column Depth $t_{ws} = 0.44\ \mathrm{in}$ - Column Web Thickness $b_{fs} = 14.5\ \mathrm{in}$ - Column Flange Width $t_{fs} = 0.71\ \mathrm{in}$ - Column Flange Thickness $A_{sup} = 26.5\ \mathrm{in}^2$ - Column Area **Column Grade Information:** $\mathrm{A992}$ - Material Grade $F_{ys} = 50\ \mathrm{ksi}$ - Column Yield Stress $F_{us} = 65\ \mathrm{ksi}$ - Column Tensile Stress $E = 29000\ \mathrm{ksi}$ - Column Modulus of Elasticity	
	Bolt Information at Column: $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325N}$ - Bolting Category $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter $F_{nt} = 90\ \mathrm{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 54\ \mathrm{ksi}$ - Bolt Nominal Shear Strength $N_s = 2$ - Number of Shear Planes HSSL - Bolt Hole Type at Angle STD - Bolt Hole Type at Support	
	Weld Information at Beam: $\mathrm{E70XX}$ - Weld Classification $W = 0.3125\ \mathrm{in}$ - Fillet Weld Size $F_{EXX} = 70\ \mathrm{ksi}$ - Filler Metal Classification Strength	
	Angle Properties: $\mathrm{L5x3\text{-}1/2x1/2}$ - Double Angle Section $d_a = 12\ \mathrm{in}$ - Angle Depth $t_a = 0.5\ \mathrm{in}$ - Double Angle Thickness $b_{a\_NSL} = 3.5\ \mathrm{in}$ - Double Angle Leg Width at Beam $b_{a\_OSL} = 5\ \mathrm{in}$ - Double Angle Leg Width at Support **Angle Material Grade:** $F_{ya} = 36\ \mathrm{ksi}$ - Double Angle Yield Stress $F_{ua} = 58\ \mathrm{ksi}$ - Double Angle Tensile Stress	
	Connection Geometry at Beam: $\mathrm{C\text{-}Shaped}$ - Weld Shape $clr = 0.5\ \mathrm{in}$ - Beam Web Clearance $kL = 3\ \mathrm{in}$ - Horizontal Weld Length at Beam $L = 12\ \mathrm{in}$ - Vertical Weld Length at Beam $e = 3\ \mathrm{in}$ - Weld Group Eccentricity at Beam **Connection Geometry at Support:** $n_r = 4$ - Number of Bolt Rows $s_r = 3\ \mathrm{in}$ - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0\ \mathrm{in}$ - Bolt Column Spacing **Distance Information at Support:** $g = 5\ \mathrm{in}$ - Gage at Support $L_{ev\_a} = 1.5\ \mathrm{in}$ - Vertical Edge Distance on Angle at Support $L_{eh\_a} = 2.65\ \mathrm{in}$ - Horizontal Edge Distance on Angle at Support $e = 0\ \mathrm{in}$ - Bolt Eccentricity at Support	

REFERENCES	CALCULATIONS	RESULTS

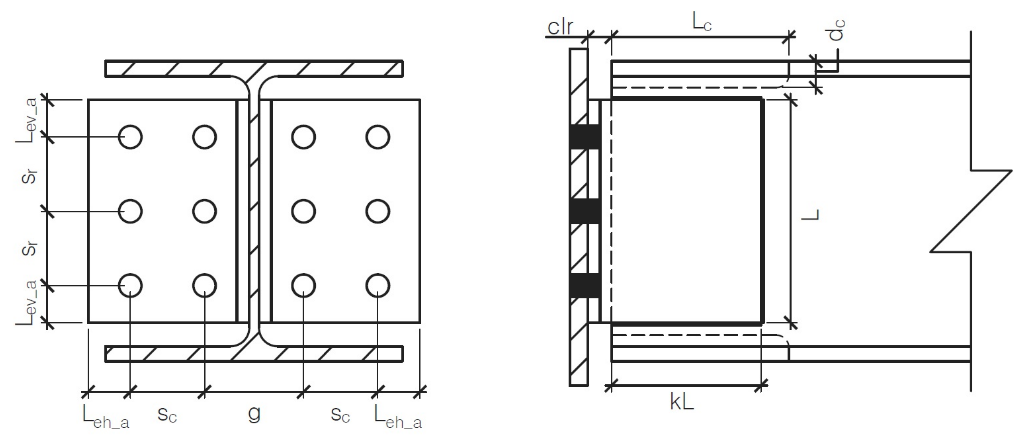

Fig. General detailing of double angle connection.

Check No. 1: Connection Detailing Limitations at Beam Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Connection Depth	7.750	12.000	0.646	**PASS**
Maximum Connection Depth	14.031	12.000	0.855	**PASS**
Maximum Fillet Weld Size per Angle Thickness	0.438	0.313	0.714	**PASS**

Result:
Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(12)}{(14.031)} = 0.85523$

PASS

Check No. 2: Design Capacity of Weld at Beam Side

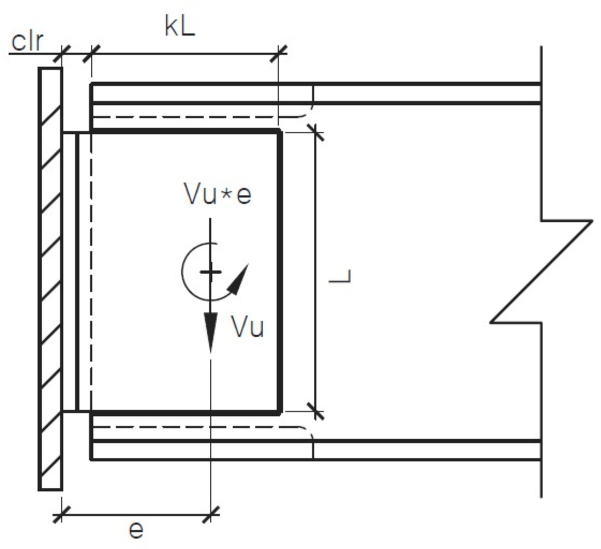

Fig. Weld group at the beam web.

Calculate the maximum fillet weld size in 16th of an inch for base metal check.

$t_a = 0.5\ \text{in}$ - Double Angle Thickness

$F_{ua} = 58\ \text{ksi}$ - Double Angle Tensile Stress

$d_{ct} = 0.15\ \text{in}$ - Cope Depth at Double Plate (assume 0)

$F_{ub} = 65\ \text{ksi}$ - Beam Tensile Stress

AISC 15th Ed. Part 9 Eq. (9-2)

D_{max} - Maximum Fillet Weld Size for Base Metal Strength

$$D_{max} = \frac{t_a\, F_{ua} \leq d_{ct}\, F_{ub}}{3.09\ \text{kip/in}}$$

$$D_{max} = \frac{(0.5\ \text{in}) \times (58\ \text{ksi}) \leq (0.15\ \text{in}) \times (65\ \text{ksi})}{(3.09\ \text{kip/in})}$$

$$D_{max} = 3.1553$$

Calculate the C-coefficient of the weld group.

$L = 12\ \text{in}$ - Vertical Weld Length at Beam

$kL = 3\ \text{in}$ - Horizontal Weld Length at Beam

$e = 3\ \text{in}$ - Weld Group Eccentricity at Beam

AISC 15th Ed. Page 8-9 to 8-12

The weld coefficient C is derived using the instantaneous center of rotation method. The coordinates of each weld segment are listed in the table below.

Weld Segment No. 1 is located at the bottom-right end of the weld group.

Segment No.	x-coordinate (in)	y-coordinate (in)
1	2.63	0.00
2	1.88	0.00
3	1.13	0.00
4	0.38	0.00
5	0.00	0.43
6	0.00	1.29
7	0.00	2.14
8	0.00	3.00
9	0.00	3.86
10	0.00	4.71
11	0.00	5.57
12	0.00	6.43
13	0.00	7.29
14	0.00	8.14
15	0.00	9.00
16	0.00	9.86
17	0.00	10.71
18	0.00	11.57
19	0.38	12.00
20	1.13	12.00
21	1.88	12.00
22	2.63	12.00

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The location of the instantaneous center with respect to the origin is calculated using iterations. The resulting coordinates of the I.C. are as follows.

$x_{ic} = -5.4 \text{ in}$ - Location of I.C. along the X-axis

$y_{ic} = 6 \text{ in}$ - Location of I.C. along the Y-axis

Using the AISC 15th Edition Equation (8-3), with standard weld size of 1/16" inches and weld electrode of FE70XX, the corresponding reactions per weld segment are listed below.

Segment No.	Ru (kip)	Rux (kip)	Ruy (kip)	Mn (kip-in)
1	1.86	1.11	1.49	18.59
2	1.85	1.18	1.43	17.45
3	1.83	1.24	1.35	16.22
4	1.80	1.29	1.24	14.95
5	2.07	1.49	1.44	16.09
6	2.00	1.31	1.51	14.33
7	1.90	1.11	1.55	12.62
8	1.79	0.87	1.56	11.04
9	1.66	0.61	1.54	9.62
10	1.52	0.35	1.48	8.42
11	1.36	0.11	1.35	7.36
12	1.36	-0.11	1.35	7.36
13	1.52	-0.35	1.48	8.42
14	1.66	-0.61	1.54	9.62
15	1.79	-0.87	1.56	11.04
16	1.90	-1.11	1.55	12.62
17	2.00	-1.31	1.51	14.33
18	2.07	-1.49	1.44	16.09
19	1.80	-1.29	1.24	14.95
20	1.83	-1.24	1.35	16.22
21	1.85	-1.18	1.43	17.45
22	1.86	-1.11	1.49	18.59

$C = 2.6554 \text{ kip/in}$ - Calculated Weld Group Coefficient

Calculate the design capacity of weld group in shear.

$C = 2.6554 \text{ kip/in}$ - Calculated Weld Group Coefficient

AISC 15th Ed. Table 8-3 $C_1 = 1$ - Electrode Strength Coefficient

$D = 5$ - Fillet Weld Size in D (in 1/16th of an inch)

AISC 15th Ed. Part 9 Eq. (9-2) $D_{max} = 3.1553$ - Maximum Fillet Weld Size for Base Metal Strength

$2L = 24 \text{ in}$ - Twice Vertical Weld Length

$\phi = 0.75$ - Fillet Weld Resistance Factor

AISC 15th Ed. Part 8 Eq. (8-21) ϕR_n - Design Strength of Weld Group

$$\phi R_n = \phi\, C\, C_1\, (D \leq D_{max})\, 2L$$

$$\phi R_n = (0.75) \times (2.6554 \text{ kip/in}) \times (1) \times ((5) \leq (3.1553)) \times (24 \text{ in})$$

$$\phi R_n = 150.82 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(30 \text{ kip})}{(150.82 \text{ kip})} = 0.19892$

PASS

Check No. 3: Design Capacity of Double Angle in Shear at Beam Side

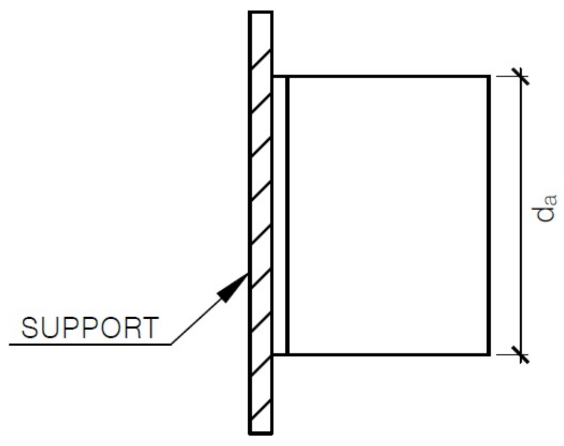

Fig. Gross and net section areas for double angle.

Calculate the gross area of double angle subject to yielding.

$d_a = 12\ \mathrm{in}$ - Angle Depth

$(2t_a) = 1\ \mathrm{in}$ - Combined Double Angle Thickness

A_{gv} - Section Gross Area

$$A_{gv} = d_a\,(2t_a)$$

$$A_{gv} = (12\ \mathrm{in}) \times (1\ \mathrm{in})$$

$$A_{gv} = 12\ \mathrm{in}^2$$

Calculate the shear yielding capacity of the double angle.

$F_{ya} = 36\ \mathrm{ksi}$ - Double Angle Yield Stress

$A_{gv} = 12\ \mathrm{in}^2$ - Section Gross Area

$\phi = 1$ - Shear Yielding Resistance Factor

AISC 360-16 Chapter J4.2 Eq. (J4-3)

ϕR_{n_sy} - Design Shear Yielding Capacity of Section

$$\phi R_{n\_sy} = \phi\,0.6\,F_{ya}\,A_{gv}$$

$$\phi R_{n\_sy} = (1) \times 0.6 \times (36\ \mathrm{ksi}) \times \left(12\ \mathrm{in}^2\right)$$

$$\phi R_{n\_sy} = 259.2\ \mathrm{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_{n\_sy}} = \frac{(30\ \mathrm{kip})}{(259.2\ \mathrm{kip})} = 0.11574$

PASS

Check No. 4: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Beam Side

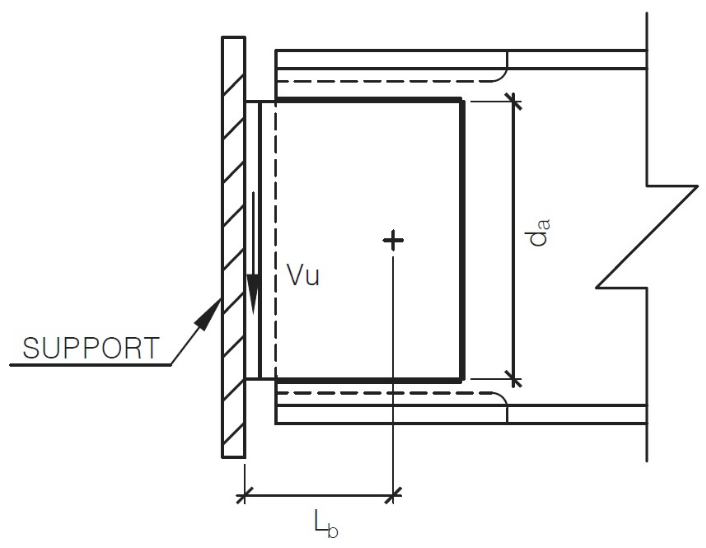

Fig. Double angle subject to flexure.

Calculate and compare upper and lower limits.

$E = 29000\ \mathrm{ksi}$ - Modulus for Steel

$F_{ya} = 36\ \mathrm{ksi}$ - Double Angle Yield Stress

$L_b = 0.5\ \mathrm{in}$ - Double Angle Unbraced Length at Beam

$d_a = 12\ \mathrm{in}$ - Angle Depth

$(2t_a) = 1\ \mathrm{in}$ - Combined Double Angle Thickness

Depth to Thickness Ratio

$\frac{L_b d}{t^2} = \frac{(0.5\ \mathrm{in})\times(12\ \mathrm{in})}{(1\ \mathrm{in})^2} = 6$

Depth to Thickness Ratio Lower Limit

$\frac{0.08E}{F_y} = \frac{0.08\times(29000\ \mathrm{ksi})}{(36\ \mathrm{ksi})} = 64.444$

Depth to Thickness Ratio Upper Limit

$\frac{1.9E}{F_y} = \frac{1.9\times(29000\ \mathrm{ksi})}{(36\ \mathrm{ksi})} = 1530.6$

So, $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$.

Calculate the plastic section modulus of the double angle.

$(2t_a) = 1\ \mathrm{in}$ - Combined Double Angle Thickness

$d_a = 12\ \mathrm{in}$ - Angle Depth

Plastic Section Modulus

$Z = \frac{(2t_a)\,(d_a)^2}{4} = \frac{(1\ \mathrm{in})\times((12\ \mathrm{in}))^2}{4} = 36\ \mathrm{in}^3$

Calculate the elastic section modulus of the double angle.

$(2t_a) = 1\ \mathrm{in}$ - Combined Double Angle Thickness

$d_a = 12\ \mathrm{in}$ - Angle Depth

Elastic Section Modulus

$S = \frac{(2t_a)\,(d_a)^2}{6} = \frac{(1\ \mathrm{in})\times((12\ \mathrm{in}))^2}{6} = 24\ \mathrm{in}^3$

Calculate the flexural capacity of the double angle.

$Z = 36\ \mathrm{in}^3$ - Plastic Section Modulus

$S = 24\ \mathrm{in}^3$ - Elastic Section Modulus

$\phi = 0.9$ - Flexure Resistance Factor

AISC 360-16 Chapter F11.1 Eq. (F11-1)

ϕM_n - Design Flexural Capacity of Coped Section

$$\phi M_n = \phi \left(F_{ya}\, Z \le 1.6\, F_{ya}\, S \right)$$

$$\phi M_n = (0.9) \times \left((36\text{ ksi}) \times \left(36\text{ in}^3\right) \le 1.6 \times (36\text{ ksi}) \times \left(24\text{ in}^3\right) \right)$$

$$\phi M_n = 97.2\text{ kipft}$$

Calculate the equivalent shear capacity of the double angle.

AISC 360-16 Chapter F11.1 Eq. (F11-1)

$\phi M_n = 97.2\text{ kipft}$ - Design Flexural Capacity of Coped Section

$L_b = 0.5\text{ in}$ - Double Angle Unbraced Length at Beam

ϕV_n - Design Capacity of Single Plate in Bending

$$\phi V_n = \frac{\phi M_n}{L_b}$$

$$\phi V_n = \frac{(97.2\text{ kipft})}{(0.5\text{ in})}$$

$$\phi V_n = 2332.8\text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi V_n} = \frac{(30\text{ kip})}{(2332.8\text{ kip})} = 0.01286$

PASS

Check No. 5: Shear and Moment Interaction of Double Angle at Beam Side

Calculate the shear and moment interaction of the double angle.

$V_r = 30\text{ kip}$ - Vertical Shear Load

Moment at Component

$M_r = V_r\, L_b = (30\text{ kip}) \times (0.5\text{ in}) = 1.25\text{ kipft}$

AISC 360-16 Chapter J4.2 Eq. (J4-3)

$\phi V_n = 259.2\text{ kip}$ - Design Shear Yielding Capacity of Section

AISC 360-16 Chapter F11.1 Eq. (F11-1)

$\phi M_n = 97.2\text{ kipft}$ - Design Flexural Capacity of Double Angle

AISC 15th Edition pg. (10-90) Eq. (10-5)

I_{nt} - Interaction for Shear Yielding/Buckling and Flexure

$$I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$$

$$I_{nt} = \left(\frac{(30\text{ kip})}{(259.2\text{ kip})} \right)^2 + \left(\frac{(1.25\text{ kipft})}{(97.2\text{ kipft})} \right)^2$$

$$I_{nt} = 0.013561$$

Result:

Demand over Capacity Ratio

$DCR = \frac{I_{nt}}{1.0} = \frac{(0.013561)}{(1)} = 0.013561$

PASS

Check No. 6: Connection Detailing Limitations at Support Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Row Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Row Spacing	6.000	3.000	0.500	PASS
Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS
Angle Minimum Horizontal Edge Distance	1.125	2.650	0.425	PASS
Maximum Connection Width	10.750	10.300	0.958	PASS
Minimum Bolt Gage for Double Angle	1.813	5.000	0.362	PASS

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(10.3)}{(10.75)} = 0.95814$

PASS

Check No. 7: Design Capacity of the Bolt Group in Shear at Support Side

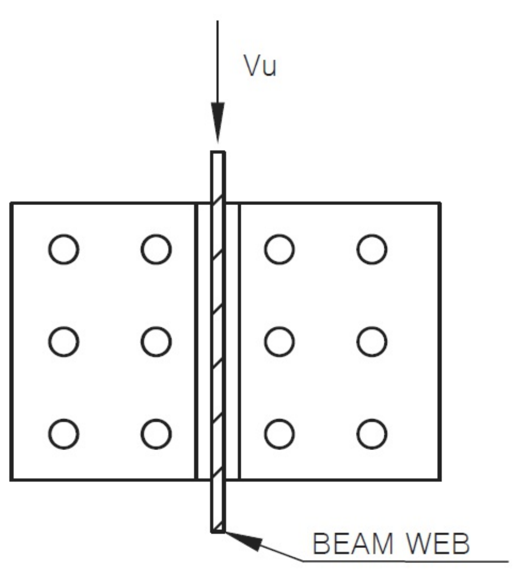

Fig. Bolt group lay-out.

Calculate the design shear capacity of the bolt group.

For bolts at Support, the full capacity of the bolt group will be considered.

Calculate the design shear capacity of the bolt group.

$F_{nv} = 54\text{ ksi}$ - Bolt Nominal Shear Strength

$d_b = 0.75\text{ in}$ - Bolt Diameter

$N_s = 1$ - Number of Shear Planes

$C = 8$ - Bolt Group Coefficient

AISC 360-16 Chapter J5.2(a)

$h_f = 1$ - Filler Factor for Bearing Bolts

$\phi = 0.75$ - Bolt Shear Resistance Factor

AISC 360-16 Chapter J3.6 Eq. (J3-1)

ϕR_n - Design Bolt Shear Capacity

$$\phi R_n = \phi \, F_{nv} \, \frac{\pi}{4} \, (d_b)^2 \, N_s \, C \, h_f$$

$$\phi R_n = (0.75) \times (54 \text{ ksi}) \times \frac{\pi}{4} \times ((0.75 \text{ in}))^2 \times (1) \times (8) \times (1)$$

$$\phi R_n = 143.14 \text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30 \text{ kip})}{(143.14 \text{ kip})} = 0.20959$

PASS

Check No. 8: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side

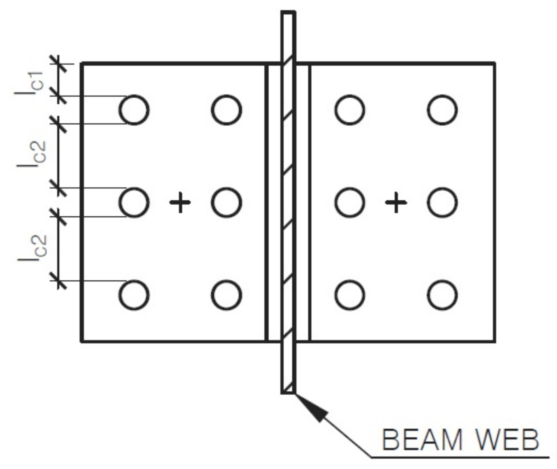

Fig. Clear distances for bolt bearing check on angle.

Calculate the bolt bearing capacity of the double angle.

$d_b = 0.75 \text{ in}$ - Bolt Diameter
$t_a = 0.5 \text{ in}$ - Double Angle Thickness
$F_{ua} = 58 \text{ ksi}$ - Double Angle Tensile Stress
$C = 8$ - Bolt Group Coefficient
$\phi = 0.75$ - Bolt Bearing Resistance Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\phi R_{n\_bearing}$ - Design Bolt Bearing Capacity of Section

$$\phi R_{n\_bearing} = \phi \, 2.4 \, d_b \, t_a \, F_{ua} \, C$$

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times (8)$$

$$\phi R_{n\_bearing} = 313.2 \text{ kip}$$

Calculate the clear distance of outer bolts on double angle.

$L_{ev\_a} = 1.5 \text{ in}$ - Vertical Edge Distance on Angle at Support
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Angle
l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev\_a} - \frac{d_h}{2}$$

$$l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$$

$$l_{c1} = 1.0938 \text{ in}$$

Calculate the clear distance of inner bolts on double angle.

$s_r = 3 \text{ in}$ - Bolt Row Spacing
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Angle
l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_r - d_h$$

$$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$$

$$l_{c2} = 2.1875 \text{ in}$$

Calculate the bolt tear-out capacity of the double angle.

$l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts
$t_a = 0.5 \text{ in}$ - Double Angle Thickness
$F_{ua} = 58 \text{ ksi}$ - Double Angle Tensile Stress
$C = 8$ - Bolt Group Coefficient
$n_r = 4$ - Number of Bolt Rows
$\phi = 0.75$ - Bolt Bearing Resistance Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\phi R_{n\_tearout}$ - Design Bolt Tear-out Capacity of Section

$$\phi R_{n\_tearout} = \phi\ [1.2\ l_{c1}\ t_a\ F_{ua} + 1.2\ l_{c2}\ t_a\ F_{ua}\ (n_r - 1)]\ \left(\frac{C}{n_r}\right)$$

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (1.0938\ \text{in}) \times (0.5\ \text{in}) \times (58\ \text{ksi}) + 1.2 \times (2.1875\ \text{in}) \times (0.5\ \text{in}) \times (58\ \text{ksi}) \times ((4) - 1)] \times \left(\frac{(8)}{(4)}\right)$$

$$\phi R_{n\_tearout} = 399.66\ \text{kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on double angle.

AISC 360-16 Chapter J3.10 Eq. (J3-6a) — $\phi R_{n\_bearing} = 313.2\ \text{kip}$ - Design bolt bearing capacity of double angle

AISC 360-16 Chapter J3.10 Eq. (J3-6c) — $\phi R_{n\_tearout} = 399.66\ \text{kip}$ - Design bolt tear-out capacity of double angle

AISC 360-16 Chapter J3.10 — ϕR_n - Governing Design Capacity

$$\phi R_n = min\left(\phi R_{n\_bearing}, \phi R_{n\_tearout}\right)$$

$$\phi R_n = min\left((313.2\ \text{kip}), (399.66\ \text{kip})\right)$$

$$\phi R_n = 313.2\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30\ \text{kip})}{(313.2\ \text{kip})} = 0.095785$

PASS

Check No. 9: Design Capacity of the Bolt Group in Bearing and Tear-out at Support Web

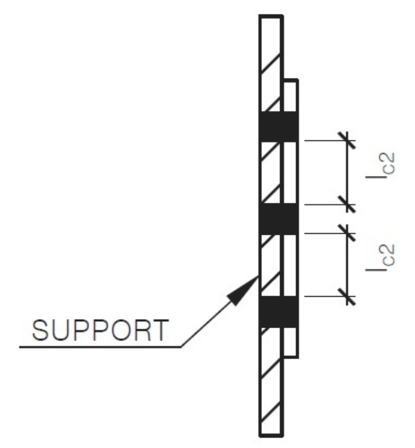

Fig. Clear distances for bolt bearing check on member.

Calculate the bolt bearing capacity of the member.

$d_b = 0.75\ \text{in}$ - Bolt Diameter

$t_{ws} = 0.44\ \text{in}$ - Column Web Thickness

$F_{ub} = 65\ \text{ksi}$ - Beam Tensile Stress

$C = 8$ - Bolt Group Coefficient

$\phi = 0.75$ - Bolt Bearing Resistance Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a) — $\phi R_{n\_bearing}$ - Design Bolt Bearing Capacity of Section

$$\phi R_{n\_bearing} = \phi\ 2.4\ d_b\ t_{ws}\ F_{ub}\ C$$

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (0.75\ \text{in}) \times (0.44\ \text{in}) \times (65\ \text{ksi}) \times (8)$$

$$\phi R_{n\_bearing} = 308.88\ \text{kip}$$

Calculate the clear distance of inner bolts on member.

$s_r = 3\ \text{in}$ - Bolt Row Spacing

$d_h = 0.8125\ \text{in}$ - Vertical Bolt Hole Dimension at Support

l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_r - d_h$$

$$l_{c2} = (3\ \text{in}) - (0.8125\ \text{in})$$

$$l_{c2} = 2.1875\ \text{in}$$

Calculate the bolt tear-out capacity of the member.

$l_{c2} = 2.1875\ \text{in}$ - Clear Distance at Rest of Bolts

$t_{ws} = 0.44\ \text{in}$ - Column Web Thickness

$F_{ub} = 65\ \text{ksi}$ - Beam Tensile Stress

$C = 8$ - Bolt Group Coefficient

$\phi = 0.75$ - Bolt Bearing Resistance Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c) — $\phi R_{n\_tearout}$ - Design Bolt Tear-out Capacity of Section

$$\phi R_{n\_tearout} = \phi\ 1.2\ l_{c2}\ t_{ws}\ F_{ub}\ C$$

$$\phi R_{n\_tearout} = (0.75) \times 1.2 \times (2.1875\ \text{in}) \times (0.44\ \text{in}) \times (65\ \text{ksi}) \times (8)$$

$$\phi R_{n\_tearout} = 450.45\ \text{kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on member.

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\phi R_{n\_bearing} = 308.88 \text{ kip}$ - Design bolt bearing capacity of member

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\phi R_{n\_tearout} = 450.45 \text{ kip}$ - Design bolt tear-out capacity of member

AISC 360-16 Chapter J3.10

ϕR_n - Governing Design Capacity

$$\phi R_n = min\left(\phi R_{n\_bearing}, \phi R_{n\_tearout}\right)$$

$$\phi R_n = min\left((308.88 \text{ kip}), (450.45 \text{ kip})\right)$$

$$\phi R_n = 308.88 \text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30 \text{ kip})}{(308.88 \text{ kip})} = 0.097125$

PASS

Check No. 10: Design Capacity of Double Angle in Block Shear at Support Side

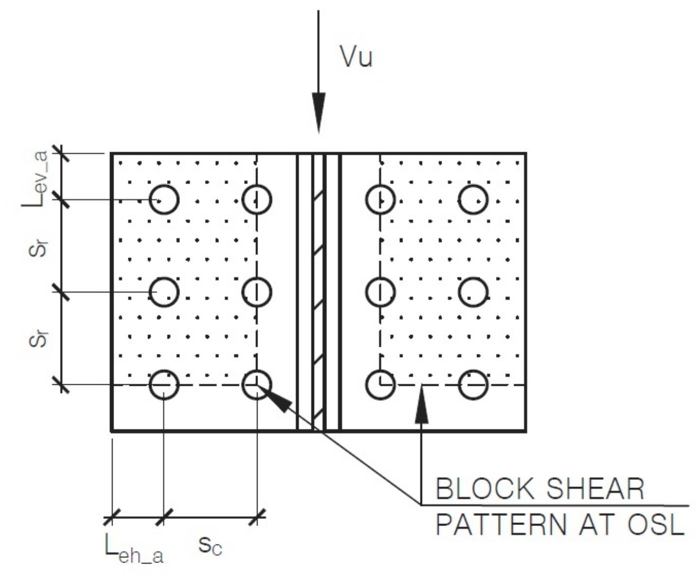

Fig. Double angle block shear pattern for shear load.

Calculate the net area of the double angle subject to tension.

$(2t_a) = 1 \text{ in}$ - Combined Double Angle Thickness
$n_c = 1$ - Number of Bolt Columns
$s_c = 0 \text{ in}$ - Bolt Column Spacing
$L_{eh\_a} = 2.65 \text{ in}$ - Horizontal Edge Distance on Angle at Support
$d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Angle
A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = (2t_a)\ \left[(n_c - 1)\ s_c + L_{eh\_a} - (n_c - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nt} = (1 \text{ in}) \times \left[((1) - 1) \times (0 \text{ in}) + (2.65 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))\right]$$

$$A_{nt} = 2.1187 \text{ in}^2$$

Calculate the gross area of the double angle subject to shear.

$(2t_a) = 1 \text{ in}$ - Combined Double Angle Thickness
$L_{ev\_a} = 1.5 \text{ in}$ - Vertical Edge Distance on Angle at Support
$n_r = 4$ - Number of Bolt Rows
$s_r = 3 \text{ in}$ - Bolt Row Spacing
A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = (2t_a)\ \left[L_{ev\_a} + (n_r - 1)\ s_r\right]$$

$$A_{gv} = (1 \text{ in}) \times \left[(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})\right]$$

$$A_{gv} = 10.5 \text{ in}^2$$

Calculate the net area of the double angle subject to shear.

$(2t_a) = 1 \text{ in}$ - Combined Double Angle Thickness
$d_a = 12 \text{ in}$ - Angle Depth
$L_{ev\_a} = 1.5 \text{ in}$ - Vertical Edge Distance on Angle at Support
$n_r = 4$ - Number of Bolt Rows
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Angle
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = (2t_a)\ \left[d_a - L_{ev\_a} - (n_r - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nv} = (1 \text{ in}) \times \left[(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))\right]$$

$$A_{nv} = 7.4375 \text{ in}^2$$

Calculate the design block shear capacity of the double angle.

$F_{ua} = 58 \text{ ksi}$ - Double Angle Tensile Stress
$A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern)
$U_{bs} = 1$ - Uniformity factor for single line of bolts
$A_{nt} = 2.1187 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{ya} = 36 \text{ ksi}$ - Double Angle Yield Stress
$A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern)
$\phi = 0.75$ - Block Shear Resistance Factor

AISC 360-16 Chapter J4.3 Eq. (J4-5)

ϕR_n - Design Block Shear Capacity of Section

$$\phi R_n = \phi\ (0.6\ F_{ua}\ A_{nv} + U_{bs}\ F_{ua}\ A_{nt} \le 0.6\ F_{ya}\ A_{gv} + U_{bs}\ F_{ua}\ A_{nt})$$

$$\phi R_n = (0.75) \times \left(0.6 \times (58 \text{ ksi}) \times \left(7.4375 \text{ in}^2\right) + (1) \times (58 \text{ ksi}) \times \left(2.1187 \text{ in}^2\right) \le 0.6 \times (36 \text{ ksi}) \times \left(10.5 \text{ in}^2\right) + (1) \times (58 \text{ ksi}) \times \left(2.1187 \text{ in}^2\right)\right)$$

$$\phi R_n = 262.27 \text{ kip}$$

Result: PASS

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30\ \text{kip})}{(262.27\ \text{kip})} = 0.11439$

Check No. 11: Design Capacity of Double Angle in Shear at Support Side

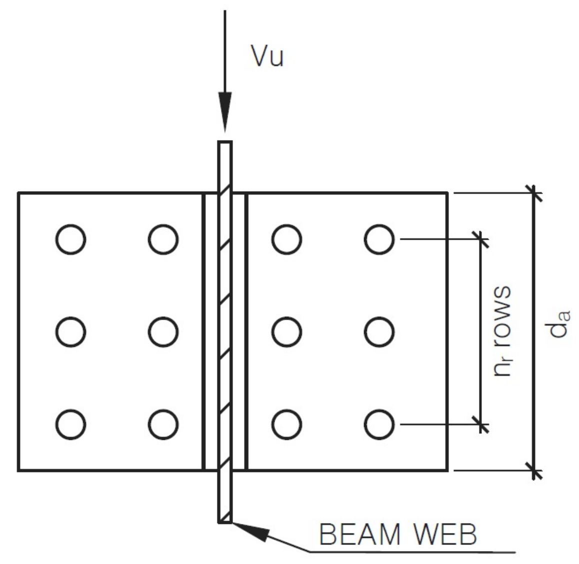

Fig. Gross and net section areas for double angle.

Calculate the gross area of double angle subject to yielding.

$d_a = 12\ \text{in}$ - Angle Depth

$(2t_a) = 1\ \text{in}$ - Combined Double Angle Thickness

A_{gv} - Section Gross Area

$$A_{gv} = d_a\,(2t_a)$$

$$A_{gv} = (12\ \text{in}) \times (1\ \text{in})$$

$$A_{gv} = 12\ \text{in}^2$$

Calculate the shear yielding capacity of the double angle.

$F_{ya} = 36\ \text{ksi}$ - Double Angle Yield Stress

$A_{gv} = 12\ \text{in}^2$ - Section Gross Area

$\phi = 1$ - Shear Yielding Resistance Factor

AISC 360-16 Chapter J4.2 Eq. (J4-3) — ϕR_{n_sy} - Design Shear Yielding Capacity of Section

$$\phi R_{n\_sy} = \phi\, 0.6\, F_{ya}\, A_{gv}$$

$$\phi R_{n\_sy} = (1) \times 0.6 \times (36\ \text{ksi}) \times \left(12\ \text{in}^2\right)$$

$$\phi R_{n\_sy} = 259.2\ \text{kip}$$

Calculate the net area of double angle subject to rupture.

$(2t_a) = 1\ \text{in}$ - Combined Double Angle Thickness

$d_a = 12\ \text{in}$ - Angle Depth

$n_r = 4$ - Number of Bolt Rows

$d_h = 0.8125\ \text{in}$ - Vertical Bolt Hole Dimension at Angle

A_{nv} - Section Net Area

$$A_{nv} = (2t_a)\ [d_a - n_r\ (d_h + 0.0625\ \text{in})]$$

$$A_{nv} = (1\ \text{in}) \times [(12\ \text{in}) - (4) \times ((0.8125\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nv} = 8.5\ \text{in}^2$$

Calculate the shear rupture capacity of the double angle.

$F_{ua} = 58\ \text{ksi}$ - Double Angle Tensile Stress

$A_{nv} = 8.5\ \text{in}^2$ - Section Net Area

$\phi = 1$ - Shear Yielding Resistance Factor

AISC 360-16 Chapter J4.2 Eq. (J4-4) — ϕR_{n_sr} - Design Shear Rupture Capacity of Section

$$\phi R_{n\_sr} = \phi\, 0.6\, F_{ua}\, A_{nv}$$

$$\phi R_{n\_sr} = (1) \times 0.6 \times (58\ \text{ksi}) \times \left(8.5\ \text{in}^2\right)$$

$$\phi R_{n\_sr} = 295.8\ \text{kip}$$

Determine the governing shear capacity of the double angle.

AISC 360-16 Chapter J4.2 Eq. (J4-3) — $\phi R_{n\_sy} = 259.2\ \text{kip}$ - Design shear yielding capacity of double angle

AISC 360-16 Chapter J4.2 Eq. (J4-4) — $\phi R_{n\_sr} = 295.8\ \text{kip}$ - Design shear rupture capacity of double angle

AISC 360-16 Chapter J4.2 — ϕR_n - Governing Design Capacity

$$\phi R_n = min\,(\phi R_{n\_sy}, \phi R_{n\_sr})$$

$$\phi R_n = min\,((259.2\ \text{kip}), (295.8\ \text{kip}))$$

$$\phi R_n = 259.2\ \text{kip}$$

Result: PASS

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30\ \text{kip})}{(259.2\ \text{kip})} = 0.11574$

Check No. 12: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side

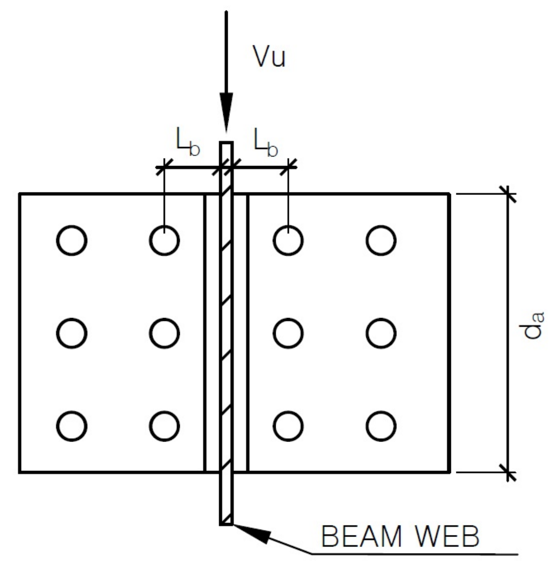

Fig. Double angle subject to flexure.

Calculate and compare upper and lower limits.

$E = 29000$ ksi - Modulus for Steel

$F_{ya} = 36$ ksi - Double Angle Yield Stress

$L_b = 2.35$ in - Double Angle Unbraced Length at Support

$d_a = 12$ in - Angle Depth

$(2t_a) = 1$ in - Combined Double Angle Thickness

Depth to Thickness Ratio

$\frac{L_b d}{t^2} = \frac{(2.35 \text{ in}) \times (12 \text{ in})}{(1 \text{ in})^2} = 28.2$

Depth to Thickness Ratio Lower Limit

$\frac{0.08E}{F_y} = \frac{0.08 \times (29000 \text{ ksi})}{(36 \text{ ksi})} = 64.444$

Depth to Thickness Ratio Upper Limit

$\frac{1.9E}{F_y} = \frac{1.9 \times (29000 \text{ ksi})}{(36 \text{ ksi})} = 1530.6$

So, $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$.

Calculate the plastic section modulus of the double angle.

$(2t_a) = 1$ in - Combined Double Angle Thickness

$d_a = 12$ in - Angle Depth

Plastic Section Modulus

$Z = \frac{(2t_a)\ (d_a)^2}{4} = \frac{(1 \text{ in}) \times ((12 \text{ in}))^2}{4} = 36 \text{ in}^3$

Calculate the elastic section modulus of the double angle.

$(2t_a) = 1$ in - Combined Double Angle Thickness

$d_a = 12$ in - Angle Depth

Elastic Section Modulus

$S = \frac{(2t_a)\ (d_a)^2}{6} = \frac{(1 \text{ in}) \times ((12 \text{ in}))^2}{6} = 24 \text{ in}^3$

Calculate the flexural capacity of the double angle.

$Z = 36 \text{ in}^3$ - Plastic Section Modulus

$S = 24 \text{ in}^3$ - Elastic Section Modulus

$\phi = 0.9$ - Flexure Resistance Factor

AISC 360-16 Chapter F11.1 Eq. (F11-1)

ϕM_n - Design Flexural Capacity of Coped Section

$$\phi M_n = \phi\,(F_{ya}\,Z \leq 1.6\,F_{ya}\,S)$$

$$\phi M_n = (0.9) \times ((36 \text{ ksi}) \times (36 \text{ in}^3) \leq 1.6 \times (36 \text{ ksi}) \times (24 \text{ in}^3))$$

$$\phi M_n = 97.2 \text{ kipft}$$

Calculate the equivalent shear capacity of the double angle.

AISC 360-16 Chapter F11.1 Eq. (F11-1)

$\phi M_n = 97.2$ kipft - Design Flexural Capacity of Coped Section

$L_b = 2.35$ in - Double Angle Unbraced Length at Support

ϕV_n - Design Capacity of Single Plate in Bending

$$\phi V_n = \frac{\phi M_n}{L_b}$$

$$\phi V_n = \frac{(97.2 \text{ kipft})}{(2.35 \text{ in})}$$

$$\phi V_n = 496.34 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi V_n} = \frac{(30 \text{ kip})}{(496.34 \text{ kip})} = 0.060442$

PASS

Check No. 13: Shear and Moment Interaction of Double Angle at Support Side

Calculate the shear and moment interaction of the double angle.

$V_r = 30$ kip - Vertical Shear Load

Moment at Component

$M_r = V_r\,L_b = (30 \text{ kip}) \times (2.35 \text{ in}) = 5.875 \text{ kipft}$

AISC 360-16 Chapter J4.2 Eq. (J4-3)

$\phi V_n = 259.2$ kip - Design shear yielding capacity of double angle

AISC 360-16 Chapter F11.1 Eq. (F11-1)

$\phi M_n = 97.2$ kipft - Design Flexural Capacity of Double Angle

AISC 15th Edition pg. (10-90) Eq. (10-5)

I_{nt} - Interaction for Shear Yielding/Buckling and Flexure

$$I_{nt} = \left(\frac{V_r}{\phi V_n}\right)^2 + \left(\frac{M_r}{\phi M_n}\right)^2$$

	$$I_{nt} = \left(\frac{(30\ \text{kip})}{(259.2\ \text{kip})}\right)^2 + \left(\frac{(5.875\ \text{kipft})}{(97.2\ \text{kipft})}\right)^2$$ $$I_{nt} = 0.017049$$ **Result:** Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.017049)}{(1)} = 0.017049$	**PASS**

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Beam Side	12.000	14.031	0.855	**PASS**
Design Capacity of Weld at Beam Side	30.000	150.817	0.199	**PASS**
Design Capacity of Double Angle in Shear at Beam Side	30.000	259.200	0.116	**PASS**
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Beam Side	30.000	2332.800	0.013	**PASS**
Shear and Moment Interaction of Double Angle at Beam Side	0.014	1.000	0.014	**PASS**
Connection Detailing Limitations at Support Side	10.300	10.750	0.958	**PASS**
Design Capacity of the Bolt Group in Shear at Support Side	30.000	143.139	0.210	**PASS**
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side	30.000	313.200	0.096	**PASS**
Design Capacity of the Bolt Group in Bearing and Tear-out at Support Web	30.000	308.880	0.097	**PASS**
Design Capacity of Double Angle in Block Shear at Support Side	30.000	262.266	0.114	**PASS**
Design Capacity of Double Angle in Shear at Support Side	30.000	259.200	0.116	**PASS**
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side	30.000	496.340	0.060	**PASS**
Shear and Moment Interaction of Double Angle at Support Side	0.017	1.000	0.017	**PASS**