AUTO DESIGN CONNECTIONS
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Subject: VB21-143

JOB: TSM1

VB21-143/VERTICAL BRACE CONNECTION: DESIGN OF SINGLE ANGLE VERTICAL BRACE TO W-SHAPE COLUMN WEB AND W-SHAPE BEAM CONNECTION

I. DESIGN DATA AND LOAD (ASD)

JOINT CODE: VB21-143

(B1010(?) BEAM TO C1101(?) COLUMN)

CONN. ID - 3169483:Vertical Brace / 3170540:Clip Angle

COLUMN PROPERTIES (col): W12X96 - A992(C1101(?))

Fy_{col} = 50 ksi	d_{col} = 12.7 in	tw_{col} = 0.55 in	$k1_{col}$ = 1.125 in
Fu_{col} = 65 ksi	bf_{col} = 12.2 in	tf_{col} = 0.9 in	k_{col} = 1.812 in
Ag_{col} = 28.2 in^2	Sx_{col} = 131 in^3	E:= 29,000 ksi	

Gage on Column, g_{col} = 5.5 in

Distance of First Bolt on Gusset from Beam Flange, y_T = 3 in

BEAM PROPERTIES (bm): W21X44 - A992(B1010(?))

Fy_{bm} = 50 ksi	d_{bm} = 20.7 in	tw_{bm} = 0.35 in	$k1_{bm}$ = 0.812 in
Fu_{bm} = 65 ksi	bf_{bm} = 6.5 in	tf_{bm} = 0.45 in	k_{bm} = 1.125 in
Ag_{bm} = 13 in^2	Sx_{bm} = 81.6 in^3		

Distance of First Bolt on Beam from Beam Flange, D = 3 in

Length of Beam, L_{bm} = 231.562 in

BRACE PROPERTIES (br): L6X4X3/8 - A36(A3000(?))

Fu_{br} = 58 ksi	ssl_{br} = 6 in	tbr = 0.375 in	Ag_{br} = 3.61 in^2
Fy_{br} = 36 ksi	osl_{br} = 4 in	$g2_{br}$ = 0 in	$g1_{br}$ = 0 in
y_{br} = 1.93 in	x_{br} = 0.933 in	g_{br} = 2.75 in	K_{br} = 1.00

Vertical Angle, θ_{br} = 53.129 deg

Length of Brace, Lu_{br} = 277.125 in

Offset from Brace, O_T = 1 in O_B = 1 in

GUSSET PLATE (gp): A36

Fy_{gp} = 36 ksi	Fu_{gp} = 58 ksi	tgp1 = 0.25 in	gap = 0.5 in

K_{wh} = 0.50

Distance of First Bolt on Brace form Working Point, x_{wp} = 24.702 in

CONNECTION ANGLE on GUSSET PLATE PROPERTIES (cag): L4X4X5/16 - A36

Fy_{cag} = 36 ksi leg_{1cag}= 4 in   tcag1 = 0.312 in n_{cag} = 2

Fu_{cag} = 58 ksi leg_{2cag}= 4 in   Column Side Bolt Gage, g_{cag} = 2.625 in

CONNECTION ANGLE on BEAM PROPERTIES (cab): L4X3-1/2X1/4 - A36

Fy_{cab} = 36 ksi leg_{1cab}= 4 in   tcab1 = 0.25 in n_{cab} = 2

Fu_{cab} = 58 ksi leg_{2cab}= 3.5 in   Beam Side Bolt Gage, g_{cab} = 2.562 in

BOLTS:

For Brace to Gusset Plate Connection:

db_{br} = 0.75 in $Bolt\_Type_{br}$ = A325N

Arv_{br}= 10.603 kips   Hole Diameter:

Arn_{br}= 19.88 kips   Brace, hd_{brv} = 0.875 in hd_{brh} = 0.875 in

Lev_{br}= 1.5 in   Gusset Plate, hd_{gpbv} = 0.875 in hd_{gpbh} = 0.875 in

Lev_{gp}= 1.5 in

s_{br} = 4 in

For Brace to Gusset Plate Connection:

number of bolt rows: $nr1_{br}$ = 4

number of vertical bolt lines: nv_{br} = 1

total number of bolts: $n_{br}= nr1_{br} \cdot nv_{br}$ n_{br} = 4

For Connection Angle on Gusset Plate to Column Connection:

db_{gp} = 0.75 in $Bolt\_Type_{gpc}$= A325N

Arv_{gp}= 10.603 kips   Hole Diameter:

Arn_{gp}= 19.88 kips   Clip Angle, hd_{gcav1}= 0.875 in hd_{gcah1}= 0.875 in

s_{gp} = 3 in   Column, hd_{colv} = 0.875 in hd_{colh} = 0.875 in

Lev_{cag}= 1.5 in

For Gusset Plate to Column Connection:

number of bolt rows: $nr1_{gp}$ = 2

number of vertical bolt lines: nv_{gp} = 1

total number of bolts: $n_{gp}= nr1_{gp} \cdot nv_{gp}$ n_{gp} = 2

For Connection Angle on Beam to Column Connection:

db_{bm} = 0.75 in $Bolt\_Type_{bmc}$ = A325N

Λrv_{bm} = 10.603 kips   Hole Diameter:

Λrn_{bm} = 19.88 kips   Clip Angle, hd_{bcav2} = 0.875 in hd_{bcah2} = 1.062 in

s_{bm} = 3 in   Column, hd_{colv} = 0.875 in hd_{colh} = 0.875 in

Lev_{cab} = 1.25 in

For Beam to Column Connection:

number of bolt rows on beam: $nr1_{bm}$ = 4

number of vertical bolt lines: nv_{bm} = 1

total number of bolts: $n_{bm} = nr1_{bm} \cdot nv_{bm}$ n_{bm} = 4

WELDS: E70xx

Fu_w = 70 ksi

	Preferred Weld Size	***Length of Weld***
Gusset Plate to Beam Flange,	w_1 = 0.25 in	Lw_1 = 32.337 in
Gusset Plate to Clip Angle	w_2 = 0.1875 in	
Clip Angle to Beam,	w_3 = 0.1875 in	

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Modification Factor, $\Lambda = \dfrac{1}{\Omega}$

	safety factor	modification factor
For tension rupture,	Ω_{tr} = 2.00	Λ_{tr} = 0.5
For tension yielding,	Ω_{ty} = 1.67	Λ_{ty} = 0.6
For compression,	Ω_c = 1.67	Λ_c = 0.6
For shear,	Ω_v = 1.67	Λ_v = 0.6
For fillet weld (shear),	Ω_{vw} = 2.00	Λ_{vw} = 0.5
For shear rupture,	Ω_{vr} = 2.00	Λ_{vr} = 0.5
For shear yielding,	Ω_{vy} = 1.50	Λ_{vy} = 0.67

For bearing,	$\Omega_{brg} = 2.00$	$\Lambda_{brg} = 0.5$
For web compression buckling,	$\Omega_{cb} = 1.67$	$\Lambda_{cb} = 0.6$
For web crippling,	$\Omega_{cr} = 2.00$	$\Lambda_{cr} = 0.5$
For web yielding,	$\Omega_{wy} = 1.50$	$\Lambda_{wy} = 0.67$
For block shear	$\Omega_{bs} = 2.00$	$\Lambda_{bs} = 0.5$
For eccentric weld,	$\Omega_{ew} = 2.00$	$\Lambda_{ew} = 0.5$
For local bending,	$\Omega_{b} = 1.67$	$\Lambda_{b} = 0.6$

APPLIED LOADS:

Brace Axial Load

	Tension	***Compression***
Given Axial Load,	$Pt_{br1} = 0$ kips	$Pc_{br1} = 0$ kips
1/2 Effective Member Strength,	$Pt_{br2} = 38.988$ kips	$Pc_{br2} = 6.796$ kips
Governing Tension or Compression, Load,	If $Pt_{br1} = 0$, $Pt_{br} = Pt_{br2}$ Otherwise, $Pt_{br} = Pt_{br1}$	If $Pc_{br1} = 0$, $Pc_{br} = Pc_{br2}$ Otherwise, $Pc_{br} = Pc_{br1}$
	$Pt_{br} = 38.988$ kips	$Pc_{br} = 6.796$ kips

Governing Axial Load,

$P_{br} = \max(Pt_{br}, Pc_{br})$

$P_{br} = 38.988$ kips

Beam Load

Shear Load, $Vu_{bm} = 50$ kips (0 % UDL)

Axial Load, $Pu_{bm} = 5$ kips

II. CALCULATIONS:

A. BOLTS ON BRACE CHECK

1. Check if Given No of Bolts are Adequate

a. *Required No of Bolts on Brace per Bolt Bearing on Brace*

$Abrg_{br} = db_{br} \cdot tbr$ $\qquad Abrg_{br} = 0.281\ in^2$

Allowable Bearing Strength using edge distance,

If $hd_{brh} = hd_{ls}$,

$Fbe = Fu_{br} \cdot min[1.0 \cdot (Lev_{br} - 0.5hd_{brv}) \cdot tbr, 2.0 \cdot Abrg_{br}]$

Otherwise,

$Fbe = Fu_{br} \cdot min[1.2 \cdot (Lev_{br} - 0.5hd_{brv}) \cdot tbr, 2.4 \cdot Abrg_{br}]$

$Fbe = 27.731\ kips$

Allowable Bearing Strength using bolt spacing,

If $hd_{brh} = hd_{ls}$,

$Fbs = Fu_{br} \cdot min[1.0 \cdot (s_{br} - hd_{brv}) \cdot tbr, 2.0 \cdot Abrg_{br}]$

Otherwise,

$Fbs = Fu_{br} \cdot min[1.2 \cdot (s_{br} - hd_{brv}) \cdot tbr, 2.4 \cdot Abrg_{br}]$

$Fbs = 39.15\ kips$

Required No of Bolts,

$$nr_{brg} = Ceil\left(\frac{\frac{P_{br}}{\Lambda brg} - Fbe}{Fbs} + 1, 1\right)$$

$nr_{brg} = 3$

RESULT = Given Number of Bolts are Adequate per Bolt Bearing

b. *Required No of Bolts on Brace per Block Shear*

Reduction Factor, $U_{bs} = 1.0$

Net Tension Area,

$Ant_{bs} = (ssl_{br} - g_{br} - 0.5hd_{brh}) \cdot tbr$

$Ant_{bs} = 1.055\ in^2$

Required Net Shear Area,

$$Anv_{req} = \frac{\frac{P_{br}}{\Lambda bs} - U_{bs} \cdot Fu_{br} \cdot Ant_{bs}}{0.6Fu_{br}} \qquad Anv_{req} = 0.483\ in^2$$

Number of Bolts per Net Shear Area,

$$nr_{brv1} = Ceil\left[\frac{\frac{Anv_{req}}{tbr} - Lev_{br} + s_{br} - 0.5hd_{brv}}{s_{br}\cdot\left(1 - \frac{hd_{brv}}{s_{br}}\right)}, 1\right]$$

$$nr_{brv1} = 2$$

Required Gross Shear Area,

$$Agv_{req} = \frac{\frac{P_{br}}{\Lambda_{bs}} - U_{bs}\cdot Fu_{br}\cdot Ant_{bs}}{0.6Fy_{br}} \qquad Agv_{req} = 0.778\ in^2$$

Number of Bolts per Gross Shear Area,

$$nr_{brv2} = Ceil\left(\frac{\frac{Agv_{req}}{tbr} - Lev_{br}}{s_{br}} + 1, 1\right)$$

$$nr_{brv2} = 2$$

Governing No of Bolts Required,

$$nr_{brv} = max(nr_{brv1}, nr_{brv2}) \qquad nr_{brv} = 2$$

RESULT = Given Number of Bolts are Adequate per Block Shear

c. *Required No of Bolts per Eccentrically Loaded Bolt Group*

Bolt centerline distance from face of support,

$$a_b = g_{br} - y_{br} \qquad a_b = 0.82\ in$$

Eccentricity distance of End Reaction from bolt line,

$$e_b = a_b \qquad e_b = 0.82\ in$$

Eccentric Load Coefficient,
(Table 7-7, AISC 13th Ed.)

$$nr_{bre} = 4$$

RESULT = Given Number of Bolts are Adequate per Eccentric Bolt Capacity

d. *Governing Number of Bolts Required for Brace*

Number of Bolts Required,

$$nr_{br1} = min(max(nr_{brg}, nr_{brv}, nr_{bre}), nrbr_{max})$$

$$nr_{br1} = 4$$

Governing Number of Bolts on Brace,

$nr_{br} = 4$

RESULT = Given Number of Bolts are Adequate

2. Eccentric Bolt Shear Capacity

(AISC 13th Ed. Chapter J, Section J3.6, pages 16.1-108 to 16.1-109)

Shear Capacity per bolt,

$Arv_{br} = 10.603$ kips

Eccentric Load Coefficient,
(Table 7-7, AISC 13th Ed.)

$C = 3.855$

Eccentric Bolt Capacity,

$Reb_{br} = C \cdot Arv_{br}$

$Reb_{br} = 40.877$ kips $\qquad P_{br} = 38.988$ kips

RESULT = Bolt Group Capacity > Force Applied, LCR = 0.954, OK

3. Check for spacing

(AISC 13th Ed. Chapter J, Section J3.3 and J3.5, pages 16.1-106 to 16.1-108)

$sb_{min} = 2\frac{2}{3} \cdot db_{br}$ $\qquad sb_{min} = 2$ in

$sb_{max} = \min(12in, 24 \cdot \min(tbr, tgp1))$ $\qquad sb_{max} = 6$ in

RESULT = s > s_min & s < s_max, OK

4. Check for edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Le_{min} = 1$ in $\qquad C_2 = 0$ in

$Lev1_{min} = Le_{min} + C_2$ $\qquad Lev1_{min} = 1$ in

$Lev1_{max} = \min(6in, 12 \cdot \min(tbr, tgp1))$ $\qquad Lev1_{max} = 3$ in

RESULT = Lev > Lev_min & Lev < Lev_max, OK

5. Check for horizontal edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Leh_{br} = ssl_{br} - g_{br}$ $\qquad Leh_{br} = 3.25$ in

$Leh_{min} = Le_{min} + C_2$ $\qquad Leh_{min} = 1$ in

$Leh_{max} = \min(6in, 12 \cdot \min(tbr, tgp1))$ $\qquad Leh_{max} = 3$ in

RESULT = Leh=3.25 > Leh_max=3, VERIFY IF ACCEPTABLE!

B. BRACE CHECK

1. Tensile Yielding Capacity of Brace

(AISC 13th Ed. Chapter D, Section D2, page 16.1-26)

$Rty_{br} = \Lambda_{ty} \cdot Fy_{br} \cdot Ag_{br}$

$Rty_{br} = 77.82$ kips $\qquad Pt_{br} = 38.988$ kips

RESULT = Tensile Yielding Capacity > Force Applied, LCR = 0.501, OK

2. Tension Rupture Capacity of Brace

(AISC 13th Ed. Chapter D, Section D2, page 16.1-26)

Net Tension Area,

$Ant = Ag_{br} - (hd_{brh} \cdot tbr)$ $\qquad Ant = 3.282\ in^2$

Length of Connection,

$Lcon = s_{br} \cdot (nr_{br} - 1)$ $\qquad Lcon = 12\ in$

Eccentricity of the Connection,

$econ = x_{br}$

Shear Lag Factor,

If $nr_{br} \geq 4$,

$U_f = 0.8$

If $nr_{br} \leq 3$,

$U_f = 0.6$

$U_f = 0.8$

Reduction Coefficient,

$$U = \max\left(1 - \frac{econ}{Lcon}, U_f\right) \qquad U = 0.922$$

Effective Net Area,

$Ae = U \cdot Ant$ $\qquad Ae = 3.027\ in^2$

Tension Rupture Capacity,

$Rtr_{br} = \Lambda_{tr} \cdot Fu_{br} \cdot Ae$

$Rtr_{br} = 87.775$ kips $\qquad Pt_{br} = 38.988$ kips

RESULT = Tensile Rupture Capacity > Force Applied, LCR = 0.444, OK

3. Compression Capacity of Brace

(AISC 13th Ed. Chapter E, Section E3, page 16.1-33)

Unsupported Length,

$Lu_{br} = 277.125\ in$

Radius of Gyration about geometric axis parallel to connected leg,

If Leg_{br} = "LLBB",

$rx_{br} = rx_{br}$

If Leg_{br} = "SLBB",

$rx_{br} = ry_{br}$

$rx_{br} = 1.93\ in$

Slenderness Ratio if connected through Longer Leg,

If $0 \leq \frac{Lu_{br}}{rx_{br}} \leq 80$,

$$Klr_{brL} = 72 + 0.75 \cdot \frac{Lu_{br}}{rx_{br}}$$

If $\frac{Lu_{br}}{rx_{br}} > 80$,

$$Klr_{brL} = \min\left(32 + 1.25 \cdot \frac{Lu_{br}}{rx_{br}}, 200\right)$$

$Klr_{brL} = 200$

Slenderness Ratio if connected through Shorter Leg,

$$Klr_{brS} = Klr_{brL} + 4 \cdot \left[\left(\frac{osl_{br}}{ssl_{br}}\right)^2 - 1\right] \qquad Klr_{brS} = 197.778$$

Slenderness Ratio,

If Leg_{br} = "LLBB",

$Klr_{br} = Klr_{brL}$

If Leg_{br} = "SLBB" And $\frac{osl_{br}}{ssl_{br}} < 1.7$,

$$Klr_{br} = \max\left(Klr_{brS}, 0.95 \cdot \frac{Lu_{br}}{ry_{br}}\right)$$

If Leg_{br} = "SLBB" And $\frac{osl_{br}}{ssl_{br}} \geq 1.7$,

Klr_{br} = "Not Applicable"

$Klr_{br} = 200$

Effective Area Factor,

$Q = 0$

Elastic Critical Buckling Stress,

$$F_e = \frac{\pi^2 \cdot E}{Klr_{br}^{\,2}} \qquad F_e = 159.955 \text{ ksi}$$

Flexural Buckling Stress,

If $Klr_{br} \leq 4.71 \cdot \sqrt{\frac{E}{Q \cdot Fy_{br}}}$,

$$F_{cr} = Q \cdot 0.658^{\frac{Q \cdot Fy_{br}}{F_e}}$$

Otherwise,

$$F_{cr} = 0.877 \cdot F_e$$

$F_{cr} = 32.764$ ksi

Compression Force Carried by the Brace,

$Rcb_{br} = \Lambda_c \cdot F_{cr} \cdot Ag_{br}$

$Rcb_{br} = 13.592$ kips $\qquad Pc_{br} = 6.796$ kips

RESULT = Compression Capacity > Force Applied, LCR = 0.5,OK

4. Bolt Bearing Capacity of Brace

(AISC 13th Ed. Chapter J, Section J3.10, page 16.1-111)

$Abrg_{br} = db_{br} \cdot tbr \qquad Abrg_{br} = 0.281 \text{ in}^2$

Allowable Bearing Strength using edge distance,

If $hd_{brh} = hd_{ls}$,

$Fbe = Fu_{br} \cdot \min[1.0 \cdot (Lev_{br} - 0.5hd_{brv}) \cdot tbr, 2.0 \cdot Abrg_{br}]$

Otherwise,

$Fbe = Fu_{br} \cdot \min[1.2 \cdot (Lev_{br} - 0.5hd_{brv}) \cdot tbr, 2.4 \cdot Abrg_{br}]$

$Fbe = 27.731$ kips

Allowable Bearing Strength using bolt spacing,

If $hd_{brh} = hd_{ls}$,

$$Fbs = Fu_{br} \cdot \min[1.0 \cdot (s_{br} - hd_{brv}) \cdot tbr, 2.0 \cdot Abrg_{br}]$$

Otherwise,

$$Fbs = Fu_{br} \cdot \min[1.2 \cdot (s_{br} - hd_{brv}) \cdot tbr, 2.4 \cdot Abrg_{br}]$$

$Fbs = 39.15$ kips

Bolt Bearing Capacity,

$$Rbrg_{br} = \Lambda_{brg} \cdot [Fbe + Fbs(nr_{br} - 1)]$$

$Rbrg_{br} = 72.591$ kips $\quad P_{br} = 38.988$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.537,OK

5. Block Shear Capacity of Brace

(AISC 13th Ed. Chapter J, Section J4.3, pages 16.1-112 to 16.1-113)

Gross Shear Area,

$$Agv = [s_{br} \cdot (nr_{br} - 1) + Lev_{br}] \cdot tbr$$

$Agv = 5.062\ in^2$

Net Shear Area,

$$Anv = [(nr_{br} - 1) \cdot s_{br} + Lev_{br} - (nr_{br} - 0.5) \cdot hd_{brv}] \cdot tbr$$

$Anv = 3.914\ in^2$

Block Shear Capacity of Beam,

$$Rbs_{br} = \Lambda_{bs} \cdot \min(0.6 \cdot Fu_{br} \cdot Anv + U_{bs} \cdot Fu_{br} \cdot Ant_{bs}, 0.6 \cdot Fy_{br} \cdot Agv + U_{bs} \cdot Fu_{br} \cdot Ant_{bs})$$

$Rbs_{br} = 85.261$ kips $\quad P_{br} = 38.988$ kips

RESULT = Block Shear Capacity > Force Applied, LCR = 0.457,OK

C. <u>GUSSET PLATE CHECK</u>

1. Check if Given Gusset Plate is Adequate

a. *Required Plate Thickness per Bolt Bearing*

Length of Allowable Bearing Strength using Edge Distance,

If $hd_{gpbh} = hd_{ls}$,

$$x_{fbe} = \min[1.0 \cdot (Lev_{gp} - 0.5hd_{gpbv}), 2.4 \cdot db_{br}]$$

Otherwise,

$$x_{fbe} = \min[1.2 \cdot (Lev_{gp} - 0.5hd_{gpbv}), 2.4 \cdot db_{br}]$$

$x_{fbe} = 1.275$ in

Length of Allowable Bearing Strength using Spacing,

If $hd_{gpbh} = hd_{ls}$,

$y_{fbs} = \min[1.0 \cdot (s_{br} - hd_{gpbv}), 2.4 \cdot db_{br}]$

Otherwise,

$y_{fbs} = \min[1.2 \cdot (s_{br} - hd_{gpbv}), 2.4 \cdot db_{br}]$

$y_{fbs} = 1.8\ in$

Required Plate Thickness,

$$tgp_{brg} = \frac{P_{br}}{\Omega_{brg} \cdot nv_{br} \cdot Fu_{gp} \cdot [x_{fbe} + y_{fbs} \cdot (nr_{br} - 1)]}$$

$tgp_{brg} = 0.201\ in$

RESULT = Given Thickness of Plate is OK per Bolt Bearing

b. *Required Plate Thickness per Tear-Out Capacity*

Required Net Area,

$$Ae_{req} = \frac{P_{br}}{\Omega_{tr} \cdot 0.6 \cdot Fu_{gp}} \qquad Ae_{req} = 2.241\ in^2$$

Required Plate Thickness,

$$tgp_{to} = \frac{Ae_{req}}{s_{br} \cdot (nr_{br} - 1) + Lev_{gp} - (nr_{br} - 0.5) \cdot hd_{gpbv}}$$

$tgp_{to} = 0.215\ in$

RESULT = Given Thickness of Plate is OK per Tear-Out Capacity

c. *Required Plate Thickness per Tension Capacity of Whitmore Section*

Total width of Whitmore Section,

$bwh = 2 \cdot \tan(30deg) \cdot (nr_{br} - 1) \cdot s_{br}$

$bwh = 13.856\ in$

Gusset Plate Length along Column Side due to Connection Length,

$Lgp_{2a} = (nr1_{br} - 1) \cdot s_{gp} + Lev_{cag} + y_T + 1in$

Gusset Plate Length along Column Side due to Geometry,

$Lgp_{2b} = [[x_{wp} + (nr_{br} - 1)s_{br} + Lev_{gp} + (g_{br} + O_T) \cdot \tan(\theta_{br})] \cdot \cos(\theta_{br})] - 0.5d_{bm}$

Gusset Plate Length along Column Side,

$Lgp_2 = \max(Lgp_{2a}, Lgp_{2b}) \qquad Lgp_2 = 9\ in$

Gusset Plate Length along Beam Side,

$$Lgp_1 = [x_{wp} + (nr_{br} - 1)\cdot s_{br} + Lev_{br}]\cdot \sin(\theta_{br}) + (Leh_{br} + O_B)\cdot \cos(\theta_{br}) - (0.5tw_{col} + gap)$$

$Lw_1 = \max(Lw_1, Lgp_1)$ $Lw_1 = 32.337\ in$

Width of Whitmore Section on Column Side,

If $\theta_{br} \le 60deg$,

$$bwh_{c1} = \max(0.5bwh - \tan(\theta_{br})\cdot x_{wp}, 0in)$$

If $\theta_{br} > 60deg$,

$$bwh_{c1} = \max\left[0.5bwh - \left(\tan(\theta_{br})\cdot x_{wp} - \frac{\frac{x_{wp}}{\cos(\theta_{br})} - Lgp_2 - 0.5d_{bm}}{\sin(\theta_{br})}\right), 0in\right]$$

$bwh_{c1} = 0\ in$

$$bwh_{c2} = \max[0.5\cdot bwh - [g_{br} + O_T + [(nr_{br} - 1)\cdot s_{br} + Lev_{br}]\cdot \tan(\gamma)], 0in]$$

$bwh_{c2} = 0\ in$

If $Lgp_2 < Lgp_{2a}$,

$bwh_c = bwh_{c2}$

If $Lgp_2 \ge Lgp_{2a}$,

$bwh_c = bwh_{c1}$

$bwh_c = 0\ in$

Width of Whitmore Section on Beam Side,

If $\theta_{br} \ge 30deg$,

$$bwh_b = \max\left[0.5bwh - \left(\tan(90deg - \theta_{br})\cdot x_{wp} - \frac{0.5d_{bm}}{\sin(\theta_{br})}\right), 0in\right]$$

If $\theta_{br} < 30deg$,

$$bwh_b = \max\left(bwh - bwh_c - \frac{Lw_1 + gap}{\cos(\theta_{br})}, 0in\right)$$

$bwh_b = 1.339\ in$

Width of Whitmore Section outside the gusset,

$bwh_o = bwh_c + bwh_b$ $bwh_o = 1.339\ in$

Available Width of Whitmore Section on the gusset plate,

$bwh_{gp} = bwh - bwh_o$ $\qquad bwh_{gp} = 12.518 \text{ in}$

Required Plate Thickness,

$$tgp_{wh} = \frac{P_{br}}{\Lambda_{ty} \cdot Fy_{gp} \cdot bwh_{gp}} \qquad tgp_{wh} = 0.144 \text{ in}$$

RESULT= Given Thickness of Plate is OK per Whitmore Section Yielding Capa

d. *Required Plate Thickness per Buckling*

Effective Length of Whitmore Section,

$$\theta_{bal} = \operatorname{atan}\left(\frac{tw_{col}}{d_{bm}}\right) \qquad \theta_{bal} = 87.204 \text{ deg}$$

If $\theta_{br} \le \theta_{bal}$,

$$Lwh_1 = x_{wp} - \frac{0.5tw_{col}}{\sin(\theta_{br})}$$

If $\theta_{br} > \theta_{bal}$,

$$Lwh_1 = x_{wp} - \frac{0.5d_{bm}}{\cos(\theta_{br})}$$

$Lwh_1 = 7.453 \text{ in}$

$$Lwh_2 = x_{wp} - \frac{0.5d_{bm}}{\cos(\theta_{br})} - [\tan(30deg) \cdot (nr_{br} - 1) \cdot s_{br}] \cdot \tan(\theta_{br})$$

$Lwh_2 = -1.785 \text{ in}$

$$\eta = [\tan(\theta_{br}) \cdot (x_{wp} - Lev\quad)] - \left[\sin(\theta_{br}) \cdot \left(\frac{x_{wp} - Lev_{br}}{\cos(\theta_{br})} - 0.5d_{bm}\right)\right]$$

If $0.5bwh > \eta$

$$Lwh_3 = x_{wp} - \frac{0.5tw_{col}}{\sin(\theta_{br})} - \frac{[\tan(30deg) \cdot (nr_{br} - 1) \cdot s_{br}]}{\tan(\theta_{br})}$$

If $0.5bwh \le \eta$

$$Lwh_3 = x_{wp} - \frac{0.5tw_{col}}{\sin(\theta_{br})} + [\tan(30deg) \cdot (nr_{br} - 1) \cdot s_{br}] \cdot \tan(\theta_{br})$$

$Lwh_3 = 12.649 \text{ in}$

$$Lwh = \max\left(\frac{Lwh_1 + Lwh_2 + Lwh_3}{3}, 0.01in\right)$$

$Lwh = 6.106 \text{ in}$

Do

Initial Thickness of Gusset Plate,

$$tgp_1 = tgp_1 + \frac{1}{16}$$

Radius of Gyration of Whitmore Section,

$$r1 = \frac{tgp_1}{\sqrt{12}}$$

Elastic Critical Buckling Stress,

$$F_{1e} = \frac{\pi^2 \cdot E}{\left(\frac{K_{wh} \cdot Lwh}{r1}\right)^2}$$

Flexural Buckling Stress,

If $\frac{K_{wh} \cdot Lwh}{r1} \le 4.71 \cdot \sqrt{\frac{E}{Fy_{gp}}}$,

$$F_{cr1} = \left(0.658^{\frac{Fy_{gp}}{F_{1e}}}\right) \cdot Fy_{gp}$$

Otherwise,

$$F_{cr1} = 0.877 \cdot F_{1e}$$

Buckling Capacity of Whitmore Section,

$$Rwhb1_{gp} = \Lambda_c \cdot F_{cr1} \cdot bwh_{gp} \cdot tgp_1$$

Loop While $Rwhb1_{gp} < Pc_{br}$

Required Plate Thickness,

$$tgp_{wbc} = tgp_1$$

$$tgp_{wbc} = 0.125 \text{ in}$$

RESULT = Given Thickness of Plate is OK per Whitmore Section Buckling Cap

e. *Governing Plate Thickness Required for the Connection*

Required Plate Thickness for the Connection,

$$tgp_{req1} = \max(tgp_{brg}, tgp_{to}, tgp_{wh}, tgp_{wbc})$$

$$tgp_{req1} = 0.25 \text{ in}$$

Governing Plate Thickness,

$tgp_{req} = 0.25$ in

RESULT = Given Thickness of Plate is Adequate

2. Bolt Bearing Capacity on Gusset Plate
(AISC 13th Ed. Chapter J, Section J3.10, page 16.1-111)

$Abrg_{gp} = db_{br} \cdot tgp_{req}$ $\qquad$ $Abrg_{gp} = 0.188\ in^2$

Allowable Bearing Strength using Edge Distance,

If $hd_{gpbh} = hd_{ls}$,

$Fbe = Fu_{gp} \cdot min[1.0 \cdot (Lev_{gp} - 0.5hd_{gpbv}) \cdot tgp_{req}, 2.0 \cdot Abrggp]$

Otherwise,

$Fbe = Fu_{gp} \cdot min[1.2 \cdot (Lev_{gp} - 0.5hd_{gpbv}) \cdot tgp_{req}, 2.4 \cdot Abrggp]$

Fbe = 18.487 kips

Allowable Bearing Strength using Spacing,

If $hd_{gpbh} = hd_{ls}$,

$Fbs = Fu_{gp} \cdot min[1.0 \cdot (s_{br} - hd_{gpbv}) \cdot tgp_{req}, 2.0 \cdot Abrg_{gp}]$

Otherwise,

$Fbs = Fu_{gp} \cdot min[1.2 \cdot (s_{br} - hd_{gpbv}) \cdot tgp_{req}, 2.4 \cdot Abrg_{gp}]$

Fbs = 26.1 kips

Bolt Bearing Capacity,

$Rbrg_{gp} = \Lambda_{brg} \cdot nv_{br} \cdot [Fbe + Fbs(nr_{br} - 1)]$

$Rbrg_{gp} = 48.394$ kips $\qquad$ $P_{br} = 38.988$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.806,OK

3. Tear-Out Capacity of Gusset Plate

Net Shear Area,

$Anv = [s_{br} \cdot (nr_{br} - 1) + Lev_{gp} - (nr_{br} - 0.5) \cdot hd_{gpbv}] \cdot tgp_{req}$

$Anv = 2.609\ in^2$

Tear-out Capacity,

$Rto_{gp} = \Lambda_{tr} \cdot 0.6 \cdot Fu_{gp} \cdot Anv$

$Rto_{gp} = 45.403$ kips $\qquad$ $P_{br} = 38.988$ kips

RESULT = Tear-out Capacity > Force Applied, LCR = 0.859,OK

4. Check Tension capacity of Whitmore Section

(AISC 13th Ed. Chapter D, Section D2, page 16.1-26)

Whitmore Section Yielding Capacity,

$$Rwhy = \Lambda_{ty} \cdot Fy_{gp} \cdot bwh_{gp} \cdot tgp_{req}$$

$Rwhy = 67.461$ kips $\qquad P_{br} = 38.988$ kips

RESULT = Tensile Yielding Capacity > Force Applied, LCR = 0.578, OK

5. Buckling Capacity of Gusset Plate

(AISC 13th Ed. Chapter E, Section E3, page 16.1-33)

Radius of Gyration of Whitmore Section,

$$r = \frac{tgp_{req}}{\sqrt{12}}$$

$r = 0.072$ in

Elastic Critical Buckling Stress,

$$F_e = \frac{\pi^2 \cdot E}{\left(\frac{K_{wh} \cdot Lwh}{r}\right)^2}$$

$F_e = 159.955$ ksi

Flexural Buckling Stress,

If $\frac{K_{wh} \cdot Lwh}{r} \le 4.71 \cdot \sqrt{\frac{E}{Fy_{gp}}}$,

$$F_{cr} = \left(0.658^{\frac{Fy_{gp}}{F_e}}\right) \cdot Fy_{gp}$$

Otherwise,

$$F_{cr} = 0.877 \cdot F_e$$

$F_{cr} = 32.764$ ksi

Buckling Capacity of Whitmore Section,

$$Rwhb_{gp} = \Lambda_c \cdot F_{cr} \cdot bwh_{gp} \cdot tgp_{req}$$

$Rwhb_{gp} = 61.519$ kips $\qquad Pc_{br} = 6.796$ kips

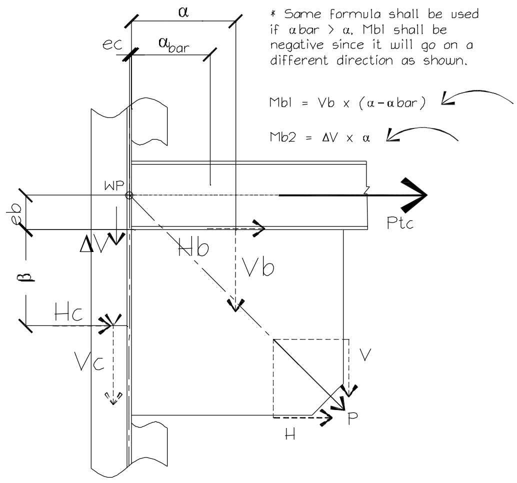
RESULT = Compression Capacity > Force Applied, LCR = 0.11,OK

D. GUSSET FORCE DISTRIBUTION (UNIFORM FORCE METHOD)

1. Calculation of Gusset Edge Forces:

(AISC, Volume II, Connections, Chapter 7, pages 7-109 to 7-111)

(Special case 2A: REDUCED VERTICAL BRACE LOAD AT BEAM, REDISTRIBUTED TO COLUMN



SPECIAL CASE 2a

$ebm = 0.5 \cdot d_{bm}$ $\qquad$ $ecol = 0in$

$\beta bar_m = 0.5 \cdot (nr1_m - 1) \cdot s_{gp} + y_T$ $\qquad$ $\alpha bar = 0.5 \cdot Lw_1 + gap$

If $\beta bar_m > \alpha bar$,

$$\beta_m = \frac{\alpha bar + ecol}{\tan(\theta_{br})} - ebm$$

Otherwise,

$$\beta_m = \beta bar_m$$

If $\alpha bar > \beta bar_m$,

$$\alpha_m = (\beta bar_m + ebm) \cdot \tan(\theta_{br}) - ecol$$

Otherwise,

$$\alpha_m = \alpha bar$$

$$r_m = \frac{P_{br}}{\sqrt{(\alpha_m + ecol)^2 + (\beta_m + ebm)^2}}$$

Horizontal side:

$H1b_m = \alpha_m \cdot r_m$

$V1b_m = ebm \cdot r_m$

$M1b_m = \left| V1b_m \cdot (\alpha_m - \alpha bar) \right|$

Vertical side:

$H1c_m = ecol \cdot r_m$

$V1c_m = \beta_m \cdot r_m$

$M1c_m = \left| H1c_m \cdot (\beta_m - \beta bar_m) \right|$

Redistribution of Forces,

$\Delta V = 0$ kips $\quad \Delta H = 0$ kips

Gusset to Beam Connection:

$Vb_m = \left| V1b_m - \Delta V \right|$

$Hb_m = H1b_m + \Delta H$

$Mb_m = \left| \Delta V \cdot \alpha_m + Vb_m \cdot (\alpha_m - \alpha bar) \right|$

Gusset to Column Connection:

$Vc_m = \left| V1c_m + \Delta V \right|$

$Hc_m = H1c_m - \Delta H$

$Mc_m = \left| Hc_m \cdot (\beta_m - \beta bar_m) \right|$

Gusset to Beam Connection:

$V_b = 16.304$ kips

$H_b = 31.19$ kips

$M_b = 51.052$ kips * in

Gusset to Column Connection:

$V_c = 7.089$ kips

$H_c = 0$ kips

$M_c = 0$ kips * in

E. <u>CONNECTION OF GUSSET PLATE TO COLUMN CHECK</u>

1. Forces Acting on Connection

Vertical Force, $V_c = 7.089$ kips

Horizontal Force, $H_c = 0$ kips

Moment Force, $M_c = 0$ kips * in

Resultant Force, $R_c = \sqrt{V_c^2 + H_c^2}$ $\qquad R_c = 7.089$ kips

2. Bolts Check

a. <u>Check if Given No of Bolts is Adequate per Vertical Force</u>

Given No of Bolts,

$nr1_{gp} = 2$

Required No of Bolts,

$nr_{greq} = 2$

Governing No of Bolts,

If Mode = Design, $\quad nr_{gp} = \max(nr_{greq1}, 2)$

If Mode = Check, $\quad nr_{gp} = nr1_{gp}$

$nr_{gp} = 2$

RESULT = Given Number of Bolts are Adequate

b. Bolt Shear Capacity

(AISC 13th Ed. Chapter J, Section J3.6, pages 16.1-108 to 16.1-109)

Shear Capacity per Bolt,

$Arv_{gp} = 10.603$ kips

Bolt Shear Capacity,

$Rb_v = n_{cag} \cdot nr_{gp} \cdot Arv_{gp}$

$Rb_v = 42.412$ kips $\qquad V_c = 7.089$ kips

RESULT = Bolt Shear Capacity > Force Applied, LCR = 0.167, OK

3. Check for Spacing

(AISC 13th Ed. Chapter J, Section J3.3 and J3.5, pages 16.1-106 to 16.1-108)

$s_{gp} = 3$ in

$sg_{min} = 2\frac{2}{3} \cdot db_{gp}$ $\qquad sg_{min} = 2$ in

$sg_{max} = \min(12in, 24 \cdot \min(tw_{col}, tcag1))$ $\qquad sg_{max} = 7.5$ in

RESULT = s > s_min & s < s_max, OK

4. Check for edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Le_{min} = 1$ in $\qquad C_2 = 0$ in

$Lev2_{min} = Le_{min} + C_2$ $\qquad Lev2_{min} = 1$ in

$Lev2_{max} = \min(6in, 12 \cdot \min(tw_{col}, tcag1))$ $\qquad Lev2_{max} = 3.75$ in

RESULT = Lev > Lev_min & Lev < Lev_max, OK

5. Check for horizontal edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Leh_{cag} = leg_{1cag} - 0.5g_{cag} - 0.5(nv_{gp} - 1) \cdot s_{gp}$

$Leh_{cag} = 1.375$ in

$Leh2_{min} = Le_{min} + C_2$ $\qquad Leh2_{min} = 1$ in

$Leh2_{max} = \min(6in, 12 \cdot \min(tw_{col}, tcag1))$ $\qquad Leh2_{max} = 3.75$ in

RESULT = Leh > Leh_min & Leh < Leh_max, OK

6. Connection Angle on Gusset Plate Check

1. *Check if Given Angle Thickness is Adequate*

a. *Required Angle Thickness on Bolt Bearing*

Length of Allowable Bearing Strength using Edge Distance,

If $hd_{gcah1} = hd_{ls}$,

$$x_{fbe} = \min[1.0\cdot(Lev_{cag} - 0.5hd_{gcav1}), 2.4\cdot db_{gp}]$$

Otherwise,

$$x_{fbe} = \min[1.2\cdot(Lev_{cag} - 0.5hd_{gcav1}), 2.4\cdot db_{gp}]$$

$x_{fbe} = 1.275$ in

Length of Allowable Bearing Strength using Spacing,

If $hd_{gcah1} = hd_{ls}$,

$$y_{fbs} = \min[1.0\cdot(s_{gp} - hd_{gcav1}), 2.4\cdot db_{gp}]$$

Otherwise,

$$y_{fbs} = \min[1.2\cdot(s_{gp} - hd_{gcav1}), 2.4\cdot db_{gp}]$$

$y_{fbs} = 1.8$ in

Required Angle Thickness,

$$tca1_{brg} = \frac{R_c}{\Lambda_{brg}\cdot n_{cag}\cdot Fu_{cag}\cdot[x_{fbe} + y_{fbs}\cdot(nr_{gp} - 1)]}$$

$tca1_{brg} = 0.04$ in

RESULT = Given Thickness of Angle is OK per Bolt Bearing

b. *Required Angle Thickness on Shear Yielding*

Length of Angle,

$$Lca = (nr_{gp} - 1)\cdot s_{gp} + 2\cdot Lev_{cag}$$

$Lca = 6$ in

Required Angle Thickness,

$$tca1_{vy} = \frac{R_c}{\Lambda_{vy}\cdot 0.6\cdot Fy_{cag}\cdot Lca\cdot n_{cag}}$$

$tca1_{vy} = 0.041$ in

RESULT = Given Thickness of Angle is OK per Shear Yielding

c. *Required Angle Thickness on Shear Rupture*

Required Angle Thickness,

$$tca1_{vr} = \frac{R_c}{\phi_{vr} \cdot 0.6 \cdot Fu_{cag} \cdot n_{cag} \cdot (Lca - nr_{gp} \cdot hd_{gcav1})}$$

$tca1_{vr} = 0.048$ in

RESULT = Given Thickness of Angle is OK per Shear Rupture

d. *Required Angle Thickness on Block Shear*

Column/ Support Side:

Reduction Factor, $U_{bs} = 1.0$

Gross Shear Length,

$Agv = n_{cag} \cdot [s_{gp} \cdot (nr_{gp} - 1) + Lev_{cag}]$ $\qquad Agv = 9$ in

Net Tension Length,

$Ant = n_{cag} \cdot (leg_{1cag} - g_{cag} - 0.5 \cdot hd_{gcah1}$ $\qquad Ant = 1.875$ in

Net Shear Length,

$Anv = n_{cag} \cdot [s_{gp} \cdot (nr_{gp} - 1) + Lev_{cag} - (nr_{gp} - 0.5) \cdot hd_{gcav1}]$

$Anv = 6.375$ in

Required Angle Thickness on Support Side,

$$tca1_{bs} = \frac{R_c}{\phi_{bs} \cdot \min(0.6Fu_{cag} \cdot Anv + U_{bs} \cdot Fu_{cag} \cdot Ant, 0.6 \cdot Fy_{cag} \cdot Agv + U_{bs} \cdot Fu_{cag} \cdot Ant)}$$

$tca1_{bs} = 0.047$ in

RESULT = Given Thickness of Angle is OK per Block Shear

e. *Angle Thickness Required per Eccentric Weld Capacity*

No. of Weld side, $n_{ws} = 2$

Length of Weld,

$Lw_{gp} = (nr_{gp} - 1) \cdot s_{gp} + 2 \cdot Lev_{cag}$ $\qquad Lw_{gp} = 6$ in

Eccentric Load Coefficient,

$kl = leg_{2cag} - gap$ $\qquad kl = 3.5$ in

$$xl = \frac{kl^2}{2kl + Lw_{gp}}$$ $\qquad xl = 0.942$ in

$al = leg_{2cag} - xl$ $\qquad al = 3.058$ in

$$k = \frac{kl}{Lw_{gp}} \qquad k = 0.583$$

$$a = \frac{al}{Lw_{gp}} \qquad a = 0.51$$

Load Inclination form vertical,

$$\theta_w = \operatorname{atan}\left(\frac{H_c}{V_c}\right) \qquad \theta_w = 0 \text{ deg}$$

Electrode Strength Coefficient,

$$C_1 = 1 \text{ ksi}$$

From Table 8-8 in AISC 13th Ed. Chapter 8,

$$C_o = 3.123$$

Required Weld Size per Shear and Axial Load,

$$w2_{req1} = \max\left(\operatorname{ceil}\left(\frac{R_c}{\phi_{ew}\cdot n_{ws}\cdot C_o\cdot C_1\cdot 16\cdot Lw_{gp}}, \frac{1}{16}\text{in}\right), \frac{3}{16}\text{in}\right)$$

$$w2_{req1} = 0.188 \text{ in}$$

Required Angle Thickness,

$$tca_{wreq} = \frac{\phi_{ew}\cdot C_o\cdot C_1\cdot 16\cdot w2_{req1}}{\phi_{vr}\cdot 0.6\cdot Fy_{cag}}$$

$$tca_{wreq} = 0.269 \text{ in}$$

RESULT = Given Thickness of Angle is OK per Eccentric Weld

f. *Required Angle Thickness per Prying Action*

Area per Bolt,

$$Ab = \frac{\pi\cdot db_{gp}^{\;2}}{4} \qquad Ab = 0.442 \text{ in}^2$$

Slip Resistance Factor,

If Code = ASD,

$$ks = 1 - \frac{1.5\cdot H_c}{1.13\cdot Tb_{gp}\cdot n_{cag}\cdot nr_{gp}}$$

If Code = LRFD,

$$ks = 1 - \frac{H_c}{1.13\cdot Tb_{gp}\cdot n_{cag}\cdot nr_{gp}}$$

$$ks = 1$$

Combined Shear & Tension Capacity per Bolt,

If $Bolt_{gp}$ = Bearing Type,

$$B_1 = Ab \cdot min\left[1.3 \cdot F_{nt2} - \left(\frac{F_{nt2}}{A_{tr} \cdot F_{nv2}}\right) \cdot \frac{V_c}{n_{cag} \cdot nr_{gp} \cdot Ab}, F_{nt2}\right]$$

If $Bolt_{gp}$ = Slip Critical,

$$B_1 = ks \cdot Arn_{gp}$$

$B_1 = 39.761$ kips

Tension Strength of Bolt Group Under Pure Tension,

$Nb1 = n_{cag} \cdot nr_{gp} \cdot Arn_{gp}$ $\qquad$ $Nb1 = 79.522$ kips

Distance from bolt centerline to the centerline of angle leg,

$b = (g_{cag} - tcag1)$ $\qquad$ $b = 2.312$ in

Distance of bolt centerline to edge of angle leg,

If $[leg_{1cag} - (b + 0.5tcag1)] \le 1.25 \cdot b$,

$$a = leg_{1cag} - (b + 0.5tcag1)$$

Otherwise,

$$a = 1.25 \cdot b$$

$a = 1.531$ in

Minimum Spacing of Bolts,

$p = s_{gp}$ $\qquad$ $p = 3$ in

$b' = b - \frac{db_{gp}}{2}$ $\qquad$ $b' = 1.938$ in

$a' = a + \frac{db_{gp}}{2}$ $\qquad$ $a' = 1.906$ in

$\rho = \frac{b'}{a'}$ $\qquad$ $\rho = 1.016$ in

$\delta = 1 - \frac{hd_{gcav1}}{p}$ $\qquad$ $\delta = 0.708$ in

$\beta = \frac{1}{\rho} \cdot \left(\frac{B}{\frac{H_c}{n_{cag} \cdot nr_{gp}}} - 1\right)$ $\qquad$ $\beta = 15646.808$ in

If $\beta \geq 1$,

$\alpha' = 1.0$

If $\beta < 1$,

$$\alpha' = \min\left[1, \frac{1}{\delta}\cdot\left(\frac{\beta}{1-\beta}\right)\right]$$

$\alpha' = 1$ in

$\lambda = 1 + \delta\cdot\alpha'$ $\lambda = 1.708$

If Code = ASD,

$$tca1_{pa} = \sqrt{\frac{6.66\cdot\left(\frac{H_c}{n_{cag}\cdot nr_{gp}}\right)\cdot b'}{p\cdot Fu_{cag}\cdot\lambda}}$$

If Code = LRFD,

$$tca1_{pa} = \sqrt{\frac{4.44\cdot\left(\frac{H_c}{n_{cag}\cdot nr_{gp}}\right)\cdot b'}{p\cdot Fu_{cag}\cdot\lambda}}$$

$tca1_{pa} = 0.01$ in

RESULT = Given Thickness of Angle is OK per Prying Action

g. Required Angle Thickness to Design the Connection

Required Thickness per above limit states,

$tca1_{req} = \max(tca1_{brg}, tca1_{vy}, tca1_{vr}, tca1_{bs}, tca1_{wreq}, tca1_{pa})$

$tca1_{req} = 0.269$ in

Governing Angle Thickness,

If Mode = Design $tca_{eq1} = tca1_{req}$

If Mode = Check $tca_{eq1} = tcag1$

$tca_{eq1} = 0.269$ in

RESULT = Required Angle Thickness is OK with Angle Size

7. Connection Angle Capacity Check

a. Bolt Bearing Capacity

(AISC 13th Ed. Chapter J, Section J3.10, page 16.1-111)

$Abrg_{ca} = db_{gp} \cdot tgp_{eq}$ $Abrg_{ca} = 0.234 \text{ in}^2$

Allowable Bearing Strength using Edge Distance,

If $hd_{gcah1} = hd_{ls}$,

$Fbe = Fu_{cag} \cdot \min[1.0 \cdot (Lev_{cag} - 0.5hd_{gcav1}) \cdot tcag_{eq}, 2.0 \cdot Abrg_{ca}]$

Otherwise,

$Fbe = Fu_{cag} \cdot \min[1.2 \cdot (Lev_{cag} - 0.5hd_{gcav1}) \cdot tcag_{eq}, 2.4 \cdot Abrg_{ca}]$

$Fbe = 23.109$ kips

Allowable Bearing Strength using Bolt Spacing,

If $hd_{gcah1} = hd_{ls}$,

$Fbs = Fu_{cag} \cdot \min[1.0 \cdot (s_{gp} - hd_{gcav1}) \cdot tcag_{eq}, 2.0 \cdot Abrg_{ca}]$

Otherwise,

$Fbs = Fu_{cag} \cdot \min[1.2 \cdot (s_{gp} - hd_{gcav1}) \cdot tcag_{eq}, 2.4 \cdot Abrg_{ca}]$

$Fbs = 32.625$ kips

Bolt Bearing Capacity,

$Rbrg_{ca} = \Lambda_{brg} \cdot nv_{cag} \cdot [Fbe + Fbs(nr_{gp} - 1)]$

$Rbrg_{ca} = 55.734$ kips $R_c = 7.089$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.127,OK

b. Shear Yielding Capacity

(AISC 13th Ed. Chapter J, Section J4.2, page 16.1-112)

Length of Angle,

$Lca = 6$ in

Gross Shear Capacity,

$Rvy_{ca} = \Lambda_{vy} \cdot 0.6 \cdot Fy_{cag} \cdot tcag_{eq} \cdot Lca \cdot n_{cag}$

$Rvy_{ca} = 54.003$ kips $R_c = 7.089$ kips

RESULT = Shear Yielding Capacity > Force Applied, LCR = 0.131,OK

c. Shear Rupture Capacity

(AISC 13th Ed. Chapter J, Section J4.2, page 16.1-112)

Net Shear Area,

$Anv = (Lca - nr_{gp} \cdot hd_{gcav1}) \cdot tcag_{eq}$ $Anv = 1.328 \text{ in}^2$

Shear Rupture Capacity,

$Rvr_{ca} = \phi_{vr} \cdot n_{cag} \cdot 0.6 \cdot Fu_{cag} \cdot Anv$

$Rvr_{ca} = 46.219$ kips $\quad R_c = 7.089$ kips

RESULT = Shear Rupture Capacity > Force Applied, LCR = 0.153,OK

d. Block Shear Capacity

(AISC 13th Ed. Chapter J, Section J4.3, page 16.1-112 to 16.1-113)

Reduction Factor, $U_{bs} = 1.00$

Gross Shear Area,

$Agv = n_{cag}[s_{gp} \cdot (nr_{gp} - 1) + Lev_{cag}] \cdot tcag_{eq}$

$Agv = 2.812\ in^2$

Net Tension Area,

$Ant = n_{cag} \cdot (leg_{1cag} - g_{cag} - 0.5 \cdot hd_{gcah1})] \cdot tcag_{eq}$

$Ant = 0.586\ in^2$

Net Shear Area,

$Anv = Agv - n_{cag} \cdot (nr_{gp} - 0.5) \cdot hd_{gcav1} \cdot tcag_{eq}$

$Anv = 1.992\ in^2$

Block Shear Capacity of Clip Angle,

$Rbs_{ca} = \phi_{bs} min(0.6 \cdot Fu_{cag} \cdot Anv + U_{bs} \cdot Fu_{cag} \cdot Ant, 0.6 \cdot Fy_{cag} \cdot Agv + U_{bs} \cdot Fu_{cag} \cdot Ant)$

$Rbs_{ca} = 47.367$ kips $\quad R_c = 7.089$ kips

RESULT = Block Shear Capacity > Force Applied, LCR = 0.15,OK

e. Bolt Tension Capacity

(AISC 13th Ed. Chapter J, Section J3.7, page 16.1-108 to 16.1-109)

Area per Bolt,

$$Ab = \frac{\pi \cdot db_{gp}^2}{4} \qquad Ab = 0.442\ in^2$$

Combined Shear & Tension Capacity per Bolt,

If $Bolt_{gp}$ = Bearing Type,

$$B_1 = Ab \cdot min\left[1.3 \cdot F_{nt2} - \left(\frac{F_{nt2}}{\phi_{tr} \cdot F_{nv2}}\right) \cdot \frac{V_c}{n_{cag} \cdot nr_{gp} \cdot Ab}, F_{nt2}\right]$$

If $Bolt_{gp}$ = Slip Critical,

$B_1 = ks \cdot Arn_{gp}$

$B_1 = 39.761$ kips

Tension Strength of Bolt Group Under Pure Tension,

$Nb1 = n_{cag} \cdot nr_{gp} \cdot Arn_{gp}$ $\qquad$ $Nb1 = 79.522$ kips

Distance from bolt centerline to the centerline of angle leg,

$b = g_{cag} - tcag_{eq}$ $\qquad$ $b = 2.312$ in

Distance of bolt centerline to edge of angle leg,

If $[leg_{1cag} - (b + 0.5tcag1)] \le 1.25 \cdot b$,

$a = leg_{1cag} - (b + 0.5tcag1)$

Otherwise,

$a = 1.25 \cdot b$

$a = 1.531$ in

Minimum Spacing of Bolts,

$p = s_{gp}$ $\qquad$ $p = 3$ in

$b' = b - \dfrac{db_{gp}}{2}$ $\qquad$ $b' = 1.938$ in

$a' = a + \dfrac{db_{gp}}{2}$ $\qquad$ $a' = 1.906$ in

$\rho = \dfrac{b'}{a'}$ $\qquad$ $\rho = 1.016$

$\delta = 1 - \dfrac{hd_{gcav1}}{p}$ $\qquad$ $\delta = 0.708$

$\beta = \dfrac{1}{\rho} \cdot \left(\dfrac{B_1}{\dfrac{H_c}{n_{cag} \cdot nr_{gp}}} - 1 \right)$ $\qquad$ $\beta = 15646.808$

If Code = ASD, $\qquad tc = \sqrt{\dfrac{6.66 \cdot B_1 \cdot b'}{p \cdot Fu_{cag}}}$

If Code = LRFD, $\qquad tc = \sqrt{\dfrac{4.44 \cdot B \cdot b'}{p \cdot Fu_{cag}}}$

$tc = 1.717$ in

$$\alpha' = \frac{1}{\delta(1+\rho)}\cdot\left[\left(\frac{tc}{tcag1}\right)^2 - 1\right] \qquad \alpha' = 20.44$$

Allowable Tensile Force of Bolt Group,

If $\alpha' > 1.0$,

$$Nb = n_{cag}\cdot nr_{gp}\cdot B_1\cdot\left(\frac{tcag_{eq}}{tc}\right)^2\cdot(1+\delta)$$

If $0 \le \alpha' \le 1.0$,

$$Nb = n_{cag}\cdot nr_{gp}\cdot B_1\cdot\left(\frac{tcag_{eq}}{tc}\right)^2\cdot(1+\delta\cdot\alpha')$$

If $\alpha' < 0$,

$$Nb = n_{cag}\cdot nr_{gp}\cdot B_1$$

$Nb = 8.998$ kips $\qquad H_c = 0.01$ kips

RESULT = Bolt Tensile Capacity > Tensile Load , LCR = 0,OK

f. Elastic Moment Capacity of the Bolt Group

Length of Column Clip Angle,

$Lca = (nr_{gp} - 1)\cdot s_{gp} + 2\cdot Lev_{cag}$ $\quad Lca = 6$ in

Assume Number of Bolt rows in Tension, $nr_{tas} = 0.5nr_{gp}$

Total Bolt Area, $Ab = \dfrac{\pi\cdot db_{gp}^2}{4}$ $\qquad Ab = 0.442\ in^2$

Assumed Total Number of Bolt rows in Tension, $n_{tas} = n_{cag}\cdot nr_{tas}$ $\quad n_{tas} = 2$

Effective Column Web Width, $w_{eff} = 8\cdot tcag_{eq} - tgp_{req}$ $\qquad w_{eff} = 1.904$ in

Number of Bolt Rows in Tension, $nr_{tas} = nr_t$

Actual Number of Bolts Under Tension,

$nr_t = 2$

Location of Neutral Axis from the bottom of Clip Angle,

$y_{cg} = 1.63$ in

Moment of Inertia of Bolts on Tension,

Distance of Bolts from the Centroid,

$y_1 = 2.87$ in

Moment of Inertia of Bolt Group in Tension,

$$Ix_{ten} = n_{cag} \cdot Ab \cdot \sum_{i = nr_{gp} - nr_t}^{nr_{gp}-1} (y_i)^2 \qquad Ix_{ten} = 7.278\ in^4$$

Outermost Bolt Location in Tension,

$ymax = 2.87\ in$

Moment of Inertia of Compressed Area,

$$Ix_{comp} = \frac{W_{eff} \cdot Y_{cg}^3}{3} \qquad Ix_{comp} = 2.749\ in^4$$

Elastic Section Modulus at Bolt Group at Tension,

$$Sx_{ten} = \frac{Ix_{ten} + Ix_{comp}}{ymax} \qquad Sx_{ten} = 3.494\ in^3$$

Equivalent Tensile Force due to Moment Load,

$$T_M = \frac{M_c}{Sx_{ten}} \cdot A_b \qquad T_M = 0\ kips$$

Total Tensile Force on Farthest Bolt,

$$T_T = \frac{H_c}{n_{cag} \cdot nr_{gp}} + T_M \qquad T_T = 0.002\ kips$$

Bolt Nominal Stress,

$F_{nt2} = 90\ kips$ $\qquad F_{nv2} = 48\ kips$

Available Tension Capacity of Bolt subject to Combined Tension and Shear,

$B_1 = 39.761\ kips$ $\qquad T_T = 0.002\ kips$

RESULT = Bolt Tensile Capacity > Force Applied, LCR = 0,OK

8. Eccentric Weld Capacity

(AISC 13th Ed. Chapter 8, pages 8-9 to 8-15)

Minimum weld size,

$w2_{min2} = 0.125\ in$ $\qquad w2 = 0.188\ in$

RESULT = Preferred Weld Size > Minimum Weld Size,OK

Shear Strength,

For Gusset Plate:

$$Rv_{gp} = A_{vr} \cdot 0.6 \cdot Fu_{gp} \cdot tgp_{req} \qquad Rv_{gp} = 4.35\ kips / in$$

For Clip Angle:

$Rv_{ca} = \Lambda_{vr} \cdot 0.6 \cdot Fu_{cag} \cdot tcag_{eq} \cdot n_{ws}$ $Rv_{ca} = 10.875$ kips / in

For Weld:

$Rv_w = \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws}$ $Rv_w = 29.698$ kips / in

Maximum effective weld size,

$$w_{eff} = \frac{\min(Rv_{gp}, Rv_{ca})}{Rv_w}$$ $w_{eff} = 0.146$ in

Governing Weld Size,

If Mode = Design, $w_{2g1} = \max(w2_{req1}, w2_{min2})$

If Mode = Check, $w_{2g1} = w2$

$w_{2g1} = 0.188$ in

Eccentric Weld Shear Capacity,

$Rew_{ca2} = \Lambda_{ew} \cdot n_{ws} \cdot C_o \cdot C_1 \cdot \min(w_{2g1}, w_{eff}) \cdot 16 \cdot Lw_{gp}$

$Rew_{ca2} = 43.919$ kips $R_c = 7.089$ kips

RESULT = Eccentric Weld Capacity > Applied Force, LCR = 0.161, OK

F. COLUMN CHECK DUE TO GUSSET STRESSES

1. Bolt Bearing Capacity

(AISC 13th Ed. Chapter J, Section J3.10, page 16.1-111)

$Abrg_{col} = db_{gp} \cdot tw_{col}$ $Abrg_{col} = 0.413$ in^2

Allowable Bearing Strength using edge distance,

If $hd_{colh} = hd_{ls}$,

$Fbe = 2.0 \cdot Abrg_{col} \cdot Fu_{col}$

Otherwise,

$Fbe = 2.4 \cdot Abrg_{col} \cdot Fu_{col}$

$Fbe = 64.35$ kips

Allowable Bearing Strength using bolt spacing,

If $hd_{colh} = hd_{ls}$,

$Fbs = Fu_{col} \cdot \min[1.0 \cdot (s_{gp} - hd_{colv}) \cdot tw_{col}, 2.0 \cdot Abrg_{col}]$

Otherwise,

$$Fbs = Fu_{col} \cdot min[1.2 \cdot (s_{gp} - hd_{colv}) \cdot tw_{col}, 2.4 \cdot Abrg_{col}]$$

$Fbs = 64.35$ kips

Bolt Bearing Capacity,

$$Rbrg_{colg} = \Lambda_{brg} \cdot 2 \cdot [Fbe + Fbs \cdot (nr_{gp} - 1)]$$

$Rbrg_{colg} = 128.7$ kips $\qquad V_c = 7.089$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.055,OK

2. Check for Column Web Bending

a. *Yield Line Theory: Reference: Yield Line of a Web Connection in Direct Tension by Richard H. Kapp, page 39, Eqs. 5)*

Distance of First Bolt to Last bolt,

$L_{yla} = (nr_{gp} - 1) \cdot s_{gp} \qquad L_{yla} = 3$ in

Width of Column Web in Bending,

$T_{col} = d_{col} - 2 \cdot tf_{col} \qquad T_{col} = 10.9$ in

Distance from inner yield lines, along the member,

$$b_1 = \frac{T_{col} - (2 \cdot g_{cag} + tgp_{req})}{2} \qquad b_1 = 2.612 \text{ in}$$

Internal Work for Web Fixed at Flanges,

$\Psi_1 = Fy_{col} \cdot tw_{col}^2 \qquad \Psi_1 = 15.125$ kips

Web Bending Capacity,

$$Rwbn_g = \Lambda_b \cdot \left(\frac{\Psi_1 \cdot L_{yla}}{b_1} + 4 \cdot \Psi_1 \cdot \sqrt{1 + \frac{2 \cdot g_{cag} + tgp_{req}}{2 \cdot b_1}} \right)$$

$Rwbn_g = 62.367$ kips $\qquad H_c = 0.01$ kips

RESULT = Web Bending Capacity > Force Applied , LCR = 0, OK

G. <u>GUSSET PLATE TO BEAM CONNECTION</u>

1. Forces Acting on Connection

Vertical Force, $V_b = 16.304$ kips

Horizontal Force, $H_b = 31.19$ kips

Moment Force, $M_b = 51.052$ kips

Resultant Force, $R_b = \sqrt{V_b^2 + H_b^2} \qquad R_b = 35.195$ kips

2. Weld of Gusset Plate to Beam

a. Using Fillet Weld

No. of Weld side, $n_{ws} = 2$

Minimum weld size,

$w1_{min1} = 0.125\ in$ $w_1 = 0.25\ in$

RESULT = Preferred Weld Size > Minimum Weld Size,OK

Maximum Force on welds per unit length,

$$fmax = \sqrt{\left(\frac{H_b}{Lw_1}\right)^2 + \left(\frac{V_b}{Lw_1} + \frac{6 \cdot M_b}{Lw_1{}^2}\right)^2}$$

$fmax = 1.251\ kips / in$

Average Force on welds per unit length,

$$fave = \frac{1}{2} \cdot \left[\sqrt{\left(\frac{H_b}{Lw_1}\right)^2 + \left(\frac{V_b}{Lw_1} - \frac{6 \cdot M_b}{Lw_1{}^2}\right)^2} + \sqrt{\left(\frac{H_b}{Lw_1}\right)^2 + \left(\frac{V_b}{Lw_1} + \frac{6 \cdot M_b}{Lw_1{}^2}\right)^2}\right]$$

$fave = 1.119\ kips / in$

Total Force per unit length on welds of Gusset Plate to Beam Conn.

$Ru_{wbm} = max(fmax, 1.25 \cdot fave)$ $Ru_{wbm} = 1.399\ kips / in$

Required Weld Size,

$$w1_{req1} = max\left(Ceil\left(\frac{Ru_{wbm}}{\Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot sin(45deg) \cdot n_{ws}}, \frac{1}{16}in\right), \frac{3}{16}in\right)$$

$w1_{req1} = 0.188\ in$

Required gusset plate thickness,

$$tgp_{r1} = \frac{\Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot sin(45deg) \cdot n_{ws} \cdot w1_{req1}}{\Lambda_{vr} \cdot 0.6 \cdot Fu_{gp}}$$

$tgp_{r1} = 0.32\ in$

Shear Strength,

For Beam:

$Rv_{bmf} = \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tf_{bm} \cdot n_{ws}$

$Rv_{bmf} = 17.55\ kips / in$

For Gusset Plate:

$Rv_{gp} = \Lambda_{vr} \cdot 0.6 \cdot Fu_{gp} \cdot tgp_{req}$ $\quad Rv_{gp} = 4.35$ kips / in

For Weld:

$Rv_w = \Lambda_{vw} \cdot 0.6 \cdot Fy_w \cdot \sin(45deg) \cdot n_{ws}$

$Rv_w = 29.698$ ksi

Maximum effective weld size,

$$w_{eff} = \frac{\min(Rv_{bmf}, Rv_{gp})}{Rv_w} \qquad w_{eff} = 0.146 \text{ in}$$

Governing Weld Size,

If Mode = Design, $w_{1g1} = \max(w1_{req1}, w1_{min1})$

If Mode = Check, $w_{1g1} = w1$

$w_{1g1} = 0.25$ in

Weld Capacity,

$Rw_{gp} = \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws} \cdot \min(w_{1g1}, w_{eff})$

$Rw_{gp} = 4.35$ kips $\quad Ru_{wbm} = 1.399$ kips

RESULT = Weld Capacity > Applied Force, LCR = 0.322, OK

b. *Using Partial Penetration Groove Weld: BTC-P4-GF*

Root Face, $f = \frac{1}{8}$ in

End Preparation,

$tgp_S = tgp_{req} - f \qquad tgp_S = 0.125$ in

Effective Weld Thickness,

$$tgp_E = \max\left(tgp_S - \frac{1}{8}\text{in}, 0.01\text{in}\right) \qquad tgp_E = 0.01 \text{ in}$$

Actual Tension Load,

$$ft_w = \max\left[\frac{V_b}{Lw_1} + \frac{6 \cdot M_b}{{Lw_1}^2}, 1.25 \cdot \left(\frac{V_b}{Lw_1} + \frac{3 \cdot M_b}{{Lw_1}^2}\right)\right]$$

$ft_w = 0.813$ kips / in

Actual Shear Load,

$$fv_w = \frac{H_b}{Lw_1} \qquad fv_w = 0.965 \text{ kips / in}$$

Required effective Throat Thickness,

$$tg1_{req} = \frac{ft_w}{\Lambda_{ty} \cdot \min(Fy_{gp}, Fy_{bm})} + \frac{fv_w}{\Lambda_{twp} \cdot 0.6 \cdot Fu_w} + 0.01in$$

$tg1_{req} = 0.091$ in

Allowable Tension Load,

$Ft_{pp} = \Lambda_{ty} \cdot \min(Fy_{gp}, Fy_{bm}) \cdot tgp_E$ $Ft_{pp} = 0.216$ kips / in

Allowable Shear Load,

$Fv_{pp} = \Lambda_{twp} \cdot 0.6 \cdot Fu_w \cdot tgp_E$ $Fv_{pp} = 0.223$ kips / in

Interaction of Equation For Partial Penetration Weld Capacity,

$$IQ_{pp2} = \frac{ft_w}{Ft_{pp}} + \frac{fv_w}{Fv_{pp}}$$

$IQ_{pp2} = 8.099$

RESULT = Please refer to Fillet Weld Capacity

c. *Using Complete Penetration Groove Weld: TC-U4b-GF*

Required Gusset Plate Thickness for CJP,

$$tg2_{req} = \frac{ft_w}{\Lambda_{ty} \cdot \min(Fy_{gp}, Fy_{bm})} + \frac{fv_w}{\Lambda_{twp} \cdot 0.6 \cdot Fu_w} + 0.01in$$

$tg2_{req} = 0.091$ in

Governing Shear Plate Thickness,

If Mode = Design, $tfp_{cjpg} = \max(tgp_{req}, tg2_{req})$

If Mode = Check, $tfp_{cjpg} = tgp_{req}$

$tgp_{cjpg} = 0.25$ in

Complete Penetration Groove Weld in Tension Capacity,

$Ft_{cp} = \Lambda_{ty} \cdot \min(Fy_{gp}, Fy_{bm}) \cdot tgp_{cjpg}$

$Ft_{cp} = 5.4$ kips / in

Complete Penetration Groove Weld Shear Capacity,

$Fv_{cp} = \Lambda_{twp} \cdot 0.6 \cdot Fu_w \cdot tgp_{cjpg}$ $Fv_{cp} = 5.565$ kips / in

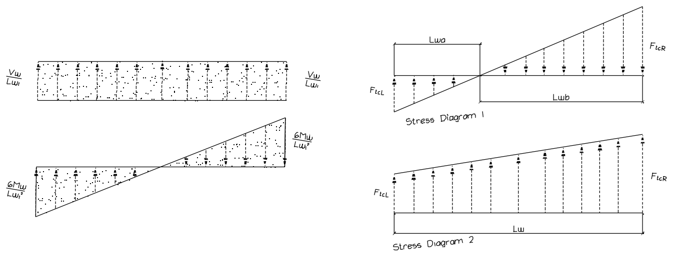
Interaction of Equation For Complete Penetration Weld Capacity,

$$IQ_{cp2} = \frac{ft_w}{Ft_{cp}} + \frac{fv_w}{Fv_{cp}}$$ $IQ_{cp2} = 0.324$

RESULT = Please refer to Fillet Weld Capacity

H. BEAM LOCAL CHECK

1. Design Forces for Local Stresses



a. Force Acting on the Beam

$$Fst_a = \frac{|V_b|}{Lw_1} - \frac{6\cdot|M_b|}{Lw_1^{\,2}} \qquad Fst_b = \frac{|V_b|}{Lw_1} + \frac{6\cdot|M_b|}{Lw_1^{\,2}}$$

$Fst_a = 0.211$ kips / in $\qquad Fst_b = 0.797$ kips / in

RESULT = Use Trapezoidal Stress Diagram

b. Design Forces Acting on the Connection

i. If Stress Diagram is Triangular

Effective Length of Weld,

$$Lw_a = \frac{|Fst_a|\cdot Lw_1}{|Fst_a|+|Fst_b|} \qquad Lw_a = 6.775 \text{ in}$$

$$Lw_b = \frac{|Fst_b|\cdot Lw_1}{|Fst_a|+|Fst_b|} \qquad Lw_b = 25.562 \text{ in}$$

Design Forces on Beam,

$$Fst_1 = \frac{1}{2}\cdot\left(|Fst_a|\cdot Lw_a\right) \qquad Fst_1 = 0.716 \text{ kips}$$

$$Fst_2 = \frac{1}{2}\cdot\left(|Fst_b|\cdot Lw_b\right) \qquad Fst_2 = 10.188 \text{ kips}$$

Maximum Design Load,

$$Fst_1 = \max(Fst_1\cdot Fst_2) \qquad Fst_1 = 10.188 \text{ kips}$$

ii. If Stress Diagram is Trapezoidal

Effective Length of Weld,

$$Lw_a = Lw_1$$

Design Forces on Beam,

$$Fst_2 = Lw_1\cdot\left(\frac{Fst_a + Fst_b}{2}\right) \qquad Fst_2 = 16.304 \text{ kips}$$

iii. Governing Design Force per Stress Diagram

Force on Gusset Plate Connecting to Beam Flange,

If $Fst_a \le 0$,

$$Fst = Fst_1$$

Otherwise,

$$Fst = Fst_2$$

$Fst = 16.304$ kips

2. Local Web Yielding

(AISC 13th Ed. Chapter J, Section J10.2, page 16.1-116)

Distance of Force to Column End,

$$De = 0.5 \cdot Lw_1$$

Bearing Length,

$$N = Lw_1$$

Web Yielding Capacity,

If $De > d_{bm}$

$$Rwy = \Lambda_{wy} [Fy_{bm} \cdot tw_{bm} \cdot (N + 5 \cdot k_{bm})$$

Otherwise

$$Rwy = \Lambda_{wy} [Fy_{bm} \cdot tw_{bm} \cdot (N + 2.5 \cdot k_{bm})$$

$Rwy = 410.094$ kips $\qquad$ $Fst = 16.304$ kips

RESULT = Web Yielding Capacity > Force Applied, LCR = 0.04, OK

3. Local Web Crippling

(AISC 13th Ed. Chapter J, Section J10.3, page 16.1-117)

Web Crippling Capacity,

$$E_{sq} = \sqrt{\frac{E \cdot Fy_{bm} \cdot tf_{bm}}{tw_{bm}}}$$

$$N_1 = 1 + 3 \cdot \left(\frac{N}{d_{bm}}\right) \cdot \left(\frac{tw_{bm}}{tf_{bm}}\right)^{1.5}$$

$$N_2 = 1 + \left(\frac{4N}{d_{bm}} - 0.2\right) \cdot \left(\frac{tw_{bm}}{tf_{bm}}\right)^{1.5}$$

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If $De \geq \frac{d_{bm}}{2}$,

$$Rwc = \phi_{cr} \cdot 0.8tw_{bm}^2 \cdot N_1 \cdot E_{sq}$$

If $De < \frac{d_{bm}}{2} \wedge \frac{N}{d_{bm}} \leq 0.2$,

$$Rwc = \phi_{cr} \cdot 0.4tw_{bm}^2 \cdot N_1 \cdot E_{sq}$$

Otherwise,

$$Rwc = \phi_{cr} \cdot 0.4tw_{bm}^2 \cdot N_2 \cdot E_{sq}$$

$Rwc = 281.975$ kips $Fst = 16.304$ kips

RESULT = Web Crippling Capacity > Applied Force, LCR = 0.058, OK

I. BEAM TO COLUMN CONNECTION

1. Forces Acring on Connection

Vertical Force, $V_{bm} = Vu_{bm} + V_b$ $V_{bm} = 66.304$ kips

Horizontal Force, $H_{bm} = Vu_{bm} + \left| P_{br} \cdot \sin(\theta_{br}) - H_b \right|$

$H_{bm} = 5$ kips

Resultant Force, $R_{bm} = \sqrt{V_{bm}^2 + H_{bm}^2}$ $R_{bm} = 66.493$ kips

2. Bolts Check

a. *Check if Given No of Bolts is Adequate per Vertical Force*

Given No of Bolts,

$nr1_{bm} = 4$ in

Required No of Bolts,

$$nr_{breq} = Ceil\left(\frac{V_{bm}}{\phi rv_{bm} \cdot n_{cab}}, 1\right)$$ $nr_{breq} = 4$ in

RESULT = Given Number of Bolts are Adequate

b. *No of Bolts Applicable in the Connection*

Maximum No of Bolts Applicable in the Connection,

$nr_{max} = 6$ in

Governing Number of Bolts,

$nr_{bm} = 4$ in

RESULT = Use Given No of Bolts

c. Bolt Shear Capacity

(AISC 13th Ed. Chapter J, Section J3.6, pages 16.1-108 to 16.1-109)

Shear Capacity per Bolt,

$Arv_{bm} = 10.603\ kips$

Bolt Shear Capacity,

$Rb_v = n_{cab} \cdot nr_{bm} \cdot Arv_{bm}$

$Rb_v = 84.823\ kips$ $V_{bm} = 66.304\ kips$

RESULT = Bolt Shear Capacity > Force Applied, LCR = 0.782, OK

3. Check for spacing

(AISC 13th Ed. Chapter J, Section J3.3 and J3.5, pages 16.1-106 to 16.1-108)

$s_{bm} = 3\ in$

$sb_{min} = 2\frac{2}{3} \cdot db$ $sb_{min} = 2\ in$

$sb_{max} = min(12in, 24 \cdot min(tw_{col}, tcab1))$ $sb_{max} = 6\ in$

RESULT = s > s_min & s < s_max, OK

4. Check for edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Le_{min} = 1\ in$ $C_2 = 0\ in$

$Lev3_{min} = Le_{min} + C_2$ $Lev3_{min} = 1\ in$

$Lev3_{max} = min(6in, 12 \cdot min(tw_{col}, tcab1))$ $Lev3_{max} = 3\ in$

RESULT = Lev > Lev_min & Lev < Lev_max, OK

5. Check for horizontal edge distance

(AISC 13th Ed. Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

$Leh_{cab} = Leg_{1cab}\ g_{cab} - 0.5(nv_{bm} - 1) \cdot s_{bm}$

$Leh_{cab} = 1.438\ in$

$Leh3_{min} = Le_{min} + C_2$ $Leh3_{min} = 1.062\ in$

$Leh3_{max} = min(6in, 12 \cdot min(tw_{col}, tcab1))$ $Leh3_{max} = 3\ in$

RESULT = Leh > Leh_min & Leh < Leh_max, OK

6. Connection Angle on Beam Check

1. Check if Given Angle Thickness is Adequate

a. Required Angle thickness on Bolt Bearing

Length of Allowable Bearing Strength using Edge Distance,

If $hd_{bcah2} = hd_{ls}$,

$x_{fbe} = min[1.0 \cdot (Lev_{cab} - 0.5hd_{bcav2}), 2.4 \cdot db_{bm}]$

Otherwise,

$x_{fbe} = \min[1.2\cdot(Lev_{cab} - 0.5hd_{bcav2}), 2.4\cdot db_{bm}]$

$x_{fbe} = 0.975$ in

Length of Allowable Bearing Strength using Spacing,

If $hd_{bcah2} = hd_{ls}$,

$y_{fbs} = \min[1.0\cdot(s_{bm} - hd_{bcav2}), 2.4\cdot db_{bm}]$

Otherwise,

$y_{fbs} = \min[1.2\cdot(s_{bm} - hd_{bcav2}), 2.4\cdot db_{bm}]$

$y_{fbs} = 1.8$ in

Required Angle Thickness,

$$tca2_{brg} = \frac{R_{bm}}{\Lambda_{brg}\cdot n_{cab}\cdot Fu_{cab}\cdot[x_{fbe} + y_{fbs}\cdot(nr_{bm} - 1)]}$$

$tca2_{brg} = 0.18$ in

RESULT = Given Thickness of Angle is OK per Bolt Bearing

b. *Required Angle Thickness on Shear Yielding*

Length of Angle,

$Lca_{bm} = (nr_{bm} - 1)\cdot s_{bm} + 2\cdot Lev_{cab}$

$Lca_{bm} = 11.5$ in

Required Angle Thickness,

$$tca2_{vy} = \frac{R_{bm}}{\Lambda_{vy}\cdot 0.6\cdot Fy_{cab}\cdot Lca_{bm}\cdot n_{cab}}$$

$tca2_{vy} = 0.201$ in

RESULT = Given Thickness of Angle is OK per Shear Yielding

c. *Required Angle Thickness on Shear Rupture*

Required Angle Thickness,

$$tca2_{vr} = \frac{R_{bm}}{\Lambda_{vr}\cdot 0.6\cdot Fu_{cab}\cdot n_{cab}\cdot(Lca_{bm} - nr_{bm}\cdot hd_{bcav2})}$$

$tca2_{vr} = 0.239$ in

RESULT = Given Thickness of Angle is OK per Shear Rupture

d. *Required Angle Thickness on Block Shear*

Column/ Support Side: *Reduction Factor,* $U_{bs} = 1.0$

Gross Shear Length,

$Agv = n_{cab} \cdot [s_{bm} \cdot (nr_{bm} - 1) + Lev_{cab}]$ $Agv = 20.5\ in$

Net Tension Length,

$Ant = n_{cab} \cdot (Leg_{1cab} - g_{cab} - 0.5 \cdot hd_{bcah2})$ $Ant = 1.812\ in$

Net Shear Length,

$Anv = n_{cab} \cdot [s_{bm} \cdot (nr_{bm} - 1) + b - (nr_{bm} - 0.5) \cdot hd_{bcav2}]$

$Anv = 14.375\ in$

Required Angle Thickness on Support Side,

$$tca2_{bs} = \frac{R_c}{\Lambda_{bs} \cdot min(0.6Fu_{cab} \cdot Anv + U_{bs} \cdot Fu_{cab} \cdot Ant, 0.6 \cdot Fy_{cab} \cdot Agv + U_{bs} \cdot Fu_{cab} \cdot Ant)}$$

$tca2_{bs} = 0.243\ in$

RESULT =

e. *Angle Thickness Required per Eccentric Weld*

No. of Weld side, $n_{ws} = 2$

Length of Weld,

$Lw_3 = (nr_{bm} - 1) \cdot s_{bm} + 2 \cdot Lev_{cab}$ $Lw_3 = 11.5\ in$

Eccentric Load Coefficient,

$kl = leg_{2cab} - gap$ $kl = 3\ in$

$xl = \frac{kl^2}{2kl + Lw_3}$ $xl = 0.514\ in$

$al = leg_{2cab} - xl$ $al = 2.986\ in$

$k = \frac{kl}{Lw_3}$ $k = 0.261$

$a = \frac{al}{Lw_3}$ $a = 0.26$

Load Inclination form vertical,

$\theta_w = atan\left(\frac{H_{bm}}{V_{bm}}\right)$ $\theta_w = 247.088\ deg$

Electrode Strength Coefficient,

$C_1 = 1$ ksi

From Table 8-8 in AISC 13th Ed. Chapter 8,

$C_o = 2.763$

Required Weld Size per Shear and Axial Load on Beam,

$$w3_{req1} = \max\left(\text{ceil}\left(\frac{R_{bm}}{\Lambda_{ew} \cdot n_{ws} \cdot C_o \cdot C_1 \cdot 16 \cdot Lw_3}, \frac{1}{16}\text{ in}\right), \frac{3}{16}\text{ in}\right)$$

$w3_{req1} = 0.188$ in

Required Angle Thickness,

$$tca_{3req} = \frac{\Lambda_{ew} \cdot C_o \cdot C_1 \cdot 16 \cdot w3_{req1}}{\Lambda_{vr} \cdot 0.6 \cdot Fy_{cab}}$$

$tca_{3req} = 0.238$ in

Maximum Angle Thickness,

$$tca_{max} = \frac{Fy_{bm}}{Fy_{cab} \cdot n_{ws}} \cdot tw_{bm}$$ $tca_{max} = 0.243$ in

Governing Angle Thickness,

If $tca_{3req} \le tca_{max}$

$tca_{3req} = tca_{3req}$

If $tca_{3req} > tca_{max}$

$tca_{3req} = tca_{max}$

$tca_{3req} = 0.238$ in

RESULT = Given Thickness of Angle is OK per Eccentric Weld

f. Required Angle Thickness per Prying Action

Area per Bolt,

$$Ab = \frac{\pi \cdot db_{bm}^{2}}{4}$$ $Ab = 0.442$ in^2

Slip Resistance Factor,

If Code = ASD,

$$ks = 1 - \frac{1.5 \cdot H_c}{1.13 \cdot Tb_{gp} \cdot n_{cag} \cdot nr_{gp}}$$

If Code = LRFD,

$$ks = 1 - \frac{H_c}{1.13 \cdot Tb_{gp} \cdot n_{cag} \cdot nr_{gp}}$$

$ks = 1$

Combined Shear & Tension Capacity per Bolt,

If $Bolt_{bm}$ = Bearing Type,

$$B_2 = Ab \cdot \min\left[1.3 \cdot F_{nt3} - \left(\frac{F_{nt3}}{\phi_{tr} \cdot F_{nv3}}\right) \cdot \frac{V_{bm}}{n_{cab} \cdot nr_{bm} \cdot Ab}, F_{nt3}\right]$$

If $Bolt_{bm}$ = Slip Critical,

$$B_2 = ks \cdot \phi rn_{bm}$$

$B = 20.609$ in

Distance from bolt centerline to the centerline of angle leg,

$b = g_{cab} - tcab1)$ $\qquad b = 2.312$ in

Distance of bolt centerline to edge of angle leg,

If $[leg_{1cab} - (b + 0.5tcab1)] \le 1.25 \cdot b$,

$$a = leg_{1cab} - (b + 0.5tcab1)$$

Otherwise,

$$a = 1.25 \cdot b$$

$a = 1.562$ in

Minimum Spacing of Bolts,

$p = s_{bm}$ $\qquad p = 3$ in

$b' = b - \frac{db_{bm}}{2}$ $\qquad b' = 1.938$ in

$a' = a + \frac{db_{bm}}{2}$ $\qquad a' = 1.938$ in

$\rho = \frac{b'}{a'}$ $\qquad \rho = 1$

$\delta = 1 - \frac{hd_{bcav2}}{p}$ $\qquad \delta = 0.708$

$\beta = \frac{1}{\rho} \cdot \left(\frac{B_2}{\frac{H_{bm}}{n_{cab} \cdot nr_{bm}}} - 1\right)$ $\qquad \beta = 16486.086$

If $\beta \ge 1$,

$$\alpha' = 1.0$$

If $\beta < 1$,

$$\alpha' = \min\left[1, \frac{1}{\delta} \cdot \left(\frac{\beta}{1 - \beta}\right)\right]$$

$\alpha' = 1$

$\lambda = 1 + \delta \cdot \alpha'$ $\lambda = 1.708$

If Code = ASD,

$$tca2_{pa} = \sqrt{\frac{6.66 \cdot \left(\frac{H_{bm}}{n_{cab} \cdot nr_{bm}}\right) \cdot b'}{p \cdot Fu_{cab} \cdot \lambda}}$$

If Code = LRFD,

$$tca2_{pa} = \sqrt{\frac{4.44 \cdot \left(\frac{H_{bm}}{n_{cab} \cdot nr_{bm}}\right) \cdot b'}{p \cdot Fu_{cab} \cdot \lambda}}$$

$tca2_{pa} = 0$ in

RESULT = Given Thickness of Angle is OK per Prying Action

g. *Governing Angle Thickness to be used*

Required Thickness of Angle to Design the Connection,

$tca2_{req} = \max(tca2_{brg}, tca2_{vy}, tca2_{vr}, tca2_{bs}, tca2_{3req}, tca2_{pa})$

$tca2_{req} = 0.243$ in

Governing Angle Thickness,

If Mode = Design $tca_{eq2} = tca2_{req}$

If Mode = Check $tca_{eq2} = tcab1$

$tca_{eq2} = 0.25$ in

RESULT = Required Angle Thickness is OK with Angle Size

7. Connection Angle Capacity Check

a. *Bolt Bearing Capacity*

(AISC 13th Ed. Chapter J, Section J3.10, page 16.1-111)

$Abrg_{ca} = db_{bm} \cdot tcab_{eq}$ $Abrg_{ca} = 0.188 \text{ in}^2$

Allowable Bearing Strength using Edge Distance,

If $hd_{bcah2} = hd_{ls}$,

$Fbe = Fu_{cab} \cdot \min[1.0 \cdot (Lev_{cab} - 0.5hd_{bcav2}) \cdot tcab_{eq}, 2.0 \cdot Abrg_{ca}]$

Otherwise,

$Fbe = Fu_{cab} \cdot \min[1.2 \cdot (Lev_{cab} - 0.5hd_{bcav2}) \cdot tcab_{eq}, 2.4 \cdot Abrg_{ca}]$

$Fbe = 14.138$ kips

Allowable Bearing Strength using Spacing,

If $hd_{bcah2} = hd_{ls}$,

$Fbs = Fu_{cab} \cdot min[1.0 \cdot (s_{bm} - hd_{bcav2}) \cdot tcab_{eq}, 2.0 \cdot Abrg_{ca}]$

Otherwise,

$Fbs = Fu_{cab} \cdot min[1.2 \cdot (s_{bm} - hd_{bcav2}) \cdot tcab_{eq}, 2.4 \cdot Abrg_{ca}]$

Fbs = 26.1 kips

Bolt Bearing Capacity,

$Rbrg_{ca} = \Lambda_{brg} \cdot n_{cab} \cdot [Fbe + Fbs(nr_{bm} - 1)]$

$Rbrg_{ca}$ = 92.438 kips $\qquad R_{bm}$ = 66.493 kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.719,OK

b. Shear Yielding Capacity

(AISC 13th Ed. Chapter J, Section J4.2, page 16.1-112)

Length of Angle,

Lca_{bm} = 11.5 in

Check if Length of Angle is acceptable per AISC requirements,

if $Lca_{bm} \geq 0.5(d_{bm} - 2k_{bm})$,

Length = "Angle Length is OK per AISC Requirements"

otherwise,

Length = "Increase Angle Length per AISC requirements"

Length = Angle Length is OK per AISC Requirements

Gross Shear Capacity,

$Rvy_{ca} = \Lambda_{vy} \cdot 0.6 \cdot Fy_{cab} \cdot tcab_{eq} \cdot Lca_{bm} \cdot n_{cab}$

Rvy_{ca} = 82.804 kips $\qquad R_{bm}$ = 66.493 kips

RESULT = Shear Yielding Capacity > Force Applied, LCR = 0.803,OK

c. Shear Rupture Capacity

(AISC 13th Ed. Chapter J, Section J4.2, page 16.1-112)

Net Shear Area,

$Anv = (Lca_{bm} - nr_{bm} \cdot hd_{bcav2}) \cdot tcab_{eq}$

$Anv = 2\ in^2$

Shear Rupture Capacity,

$$Rvr_{ca} = \Phi_{vr} \cdot n_{cab} \cdot 0.6 \cdot Fu_{cab} \cdot Anv$$

$Rvr_{ca} = 69.6$ kips $\quad R_{bm} = 66.493$ kips

RESULT = Shear Rupture Capacity > Force Applied, LCR = 0.955,OK

d. Block Shear Capacity

(AISC 13th Ed. Chapter J, Section J4.3, page 16.1-112 to 16.1-113)

Reduction Factor, $U_{bs} = 1.00$

Gross Shear Area,

$$Agv = n_{cab}[s_{bm}\ -1\ ) + Lev_{cab}] \cdot tcab_{eq}$$

$Agv = 5.125\ in^2$

Net Tension Area,

$$Ant = [n_{cab} \cdot (leg_{1cab} - g_{cab} - hd_{bcah2})] \cdot tcab_{eq}$$

$Ant = 0.453\ in^2$

Net Shear Area,

$$Anv = Agv - n_{cab} \cdot (nr_{bm} - 0.5) \cdot hd_{bcav2} \cdot tcab_{eq}$$

$Anv = 3.594\ in^2$

Block Shear Capacity of Clip Angle,

$$Rbs_{ca} = \Phi_{bs} min(0.6 \cdot Fu_{cab} \cdot Anv + U_{bs} \cdot Fu_{cab} \cdot Ant, 0.6 \cdot Fy_{cab} \cdot Agv + U_{bs} \cdot Fu_{cab} \cdot Ant)$$

$Rbs_{ca} = 68.491$ kips $\quad R_{bm} = 66.493$ kips

RESULT = Block Shear Capacity > Force Applied, LCR = 0.971,OK

e. Bolt Tension Capacity

(AISC 13th Ed. Chapter J, Section J3.7, page 16.1-108 to 16.1-109)

Area per Bolt,

$$Ab = \frac{\pi \cdot db_{bm}^{\ 2}}{4} \qquad Ab = 0.442\ in^2$$

Combined Shear & Tension Capacity per Bolt,

IF $Bolt_{bm}$ = Bearing Type,

$$B_2 = Ab \cdot min\left[1.3 \cdot F_{nt3} - \left(\frac{F_{nt3}}{\Phi_{tr} \cdot F_{nv3}}\right) \cdot \frac{V_c}{n_{cab} \cdot nr_{bm} \cdot Ab}, F_{nt3}\right]$$

IF $Bolt_{bm}$ = Slip-Critical,

$$B_2 = ks \cdot Arn_{bm}$$

$B = 20.609$ in

Distance from bolt centerline to the centerline of angle leg,

$$b = 0.5(g_{col} - tw_{bm} \quad tcab_{eq}) \qquad b = 2.312 \text{ in}$$

Distance from bolt centerline to the centerline of angle leg,

If $[leg_{1cab} - (b + 0.5tcab_{eq})] \le 1.25 \cdot b$,

$$a = leg_{1cab} - (b + 0.5tcab_{eq})$$

Otherwise,

$$a = 1.25 \cdot b$$

$a = 1.562$ in

Minimum Spacing of Bolts,

$$p = s_{bm} \qquad p = 3 \text{ in}$$

$$b' = b - \frac{db_{bm}}{2} \qquad b' = 1.938 \text{ in}$$

$$a' = a + \frac{db_{bm}}{2} \qquad a' = 1.938 \text{ in}$$

$$\rho = \frac{b'}{a'} \qquad \rho = 1$$

$$\delta = 1 - \frac{hd_{bcav2}}{p} \qquad \delta = 0.708$$

$$\beta = \frac{1}{\rho} \cdot \left(\frac{B_2}{\frac{H_{bm}}{n_{cab} \cdot nr_{bm}}} - 1 \right) \qquad \beta = 16486.086$$

If Code = ASD,

$$tc = \sqrt{\frac{6.66 \cdot B_2 \cdot b'}{p \cdot Fu_{cab}}}$$

If Code = LRFD,

$$tc = \sqrt{\frac{4.44 \cdot B_2 \cdot b'}{p \cdot Fu_{cab}}}$$

$tc = 1.236$

$$\alpha' = \frac{1}{\delta \cdot (1 + \rho)} \cdot \left[\left(\frac{tc}{tcab1} \right)^2 - 1 \right] \qquad \alpha' = 16.555$$

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Allowable Tensile Force of Bolt Group,

If α' > 1.0,

$$T_{allow} = n_{cab} \cdot nr_{bm} \cdot B_2 \cdot \left(\frac{tcab_{eq}}{tc}\right)^2 \cdot (1 + \delta)$$

If 0 ≤ α' ≤ 1.0,

$$T_{allow} = n_{cab} \cdot nr_{bm} \cdot B_2 \cdot \left(\frac{tcab_{eq}}{tc}\right)^2 \cdot (1 + \delta \cdot \alpha')$$

If α' < 0,

$$T_{allow} = n_{cab} \cdot nr_{bm} \cdot B_2 \cdot$$

$T_{allow} = 11.518$ kips $\quad H_{bm} = 0$ kips

RESULT = Bolt Tensile Capacity w/ Prying of Angle > Tensile Load, LCR = 0,OK

8. Weld Capacity of Angle on Beam Web

(AISC 13th Ed. Chapter 8, pages 8-9 to 8-15)

Minimum weld size,

$w3_{min1} = 0.125$ in $\quad w_3 = 0.188$ in

RESULT = Preferred Weld Size > Minimum Weld Size,OK

Shear Strength,

For Beam:

$Rv_{bm2} = \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tw_{bm}$ $\quad Rv_{bm2} = 6.825$ kips / in

For Clip Angle:

$Rv_{ca} = \Lambda_{vr} \cdot 0.6 \cdot Fu_{cab} \cdot tcab_{eq} \cdot n_{ws}$ $\quad Rv_{ca} = 8.7$ kips / in

For Weld:

$Rv_w = \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws}$ $\quad Rv_w = 29.698$ ksi

Maximum effective weld size,

$$w_{eff} = \frac{\min(Rv_{bm2}, Rv_{ca})}{Rv_w}$$ $w_{eff} = 0.23$ in

Governing Weld Size,

If Mode = Design, $\quad w_{3g1} = \max(w3_{req1}, w3_{min1})$

If Mode = Check, $\quad w_{3g1} = w_3$

$w_{3g1} = 0.188$ in

Eccentric Weld Shear Capacity,

$$Rew_{ca3} = \Lambda_{ew} \cdot n_{ws} \cdot C_o \cdot C_1 \cdot \min(w_{3g1}, w_{eff}) \cdot 16 \cdot Lw_3$$

$Rew_{ca3} = 95.314$ kips $\qquad R_{bm} = 66.493$ kips

RESULT = Eccentric Weld Capacity > Applied Force, LCR = 0.698, OK

9. Beam Check

a. Shear Capacity of Beam

(AISC 13th Ed. Cgapter G, Section G2.1, pages 16.1-64 to 16.1-66)

Clear distance between flanges of beam,

$h = d_{bm} - 2 \cdot k_{bm}$ $\qquad h = 18.45$ in

Limiting ratio,

$$h_{tw} = \frac{h}{tw_{bm}}$$ $\qquad h_{tw} = 52.714$ in

Determine if Intermediate stiffeners are required,

If $h_{tw} \le 260$, $\qquad a = 0$ in

Otherwise, $\qquad a = \min\left[3 \cdot h, \frac{(260 \cdot tw_{bm})^2}{h}\right]$

$a = 0$ in

RESULT = Intermediate Stiffeners NOT REQUIRED

Web plate buckling coefficient,

$kv = 5$

Web shear coefficient,

$Cv = 1$

Shear Capacity of Section,

$$Rv_{bm} = \Lambda_{vbm} \cdot 0.6 \cdot Fy_{bm} \cdot d_{bm} \cdot tw_{bm} \cdot Cv$$

$Rv_{bm} = 144.907$ kips $\qquad Vu_{bm} = 50$ kips

RESULT = Shear Yielding Capacity > Force Applied, LCR = 0.345,OK

10. Column Check

a. Bolt Bearing Capacity

(AISC 13th Ed. Cgapter J, Section J3.10, pages 16.1-111)

Effective Column Web Thickness,

If $Vu_{bm2} > 0$ kips,

$$tw_{cole} = tw_{col} \cdot \left(\frac{V_{bm}}{V_{bm} + Vu_{bm2}}\right)$$

Otherwise,

$tw_{cole} = tw_{col}$

$tw_{cole} = 0.55\ in$

$Abrg_{col} = db_{gp} \cdot tw_{cole}$ $\qquad Abrg_{col} = 0.413\ in^2$

Allowable Bearing Strength using edge distance,

If $hd_{colh} = hd_{ls}$,

$Fbe = 2.0 \cdot Abrg_{col} \cdot Fu_{col}$

Otherwise,

$Fbe = 2.4 \cdot Abrg_{col} \cdot Fu_{col}$

$Fbe = 64.35\ kips$

Allowable Bearing Strength using bolt spacing,

If $hd_{colh} = hd_{ls}$,

$Fbs = Fu_{col} \cdot \min[1.0 \cdot (s_{bm} - hd_{colv}) \cdot tw_{cole}, 2.0 \cdot Abrg_{col}]$

Otherwise,

$Fbs = Fu_{col} \cdot \min[1.2 \cdot (s_{bm} - hd_{colv}) \cdot tw_{cole}, 2.4 \cdot Abrg_{col}]$

$Fbs = 64.35\ kips$

Bolt Bearing Capacity,

$Rbrg_{col} = \Lambda_{brg} \cdot 2 \cdot [Fbe + Fbs \cdot (nr_{bm} - 1)]$

$Rbrg_{col} = 0\ kips$ $\qquad V_{bm} = 66.304\ kips$

RESULT =

b. Check for Column Web Bending

Yield Line Theory: Reference: Yield Line of a Web Connection in Direct Tension by Richard H. Kapp, page 39, Eqs. 5)

Distance of First Bolt to Last bolt,

$L_{yla} = (nr_{bm} - 1) \cdot s_{bm}$ $\qquad L_{yla} = 9\ in$

Width of Column Web in Bending,

$T_{col} = d_{col} - 2 \cdot tf_{col}$ $\qquad T_{col} = 10.9\ in$

Distance from inner yield lines, along the member,

$b_1 = \dfrac{T_{col} - (2 \cdot g_{cab} + tw_{bm})}{2}$ $\qquad b_1 = 2.612\ in$

Internal Work for Web Fixed at Flanges,

$\psi_1 = Fy_{col} \cdot tw_{col}^2$ $\quad\quad$ $\psi_1 = 15.125$ kips

Web Bending Capacity,

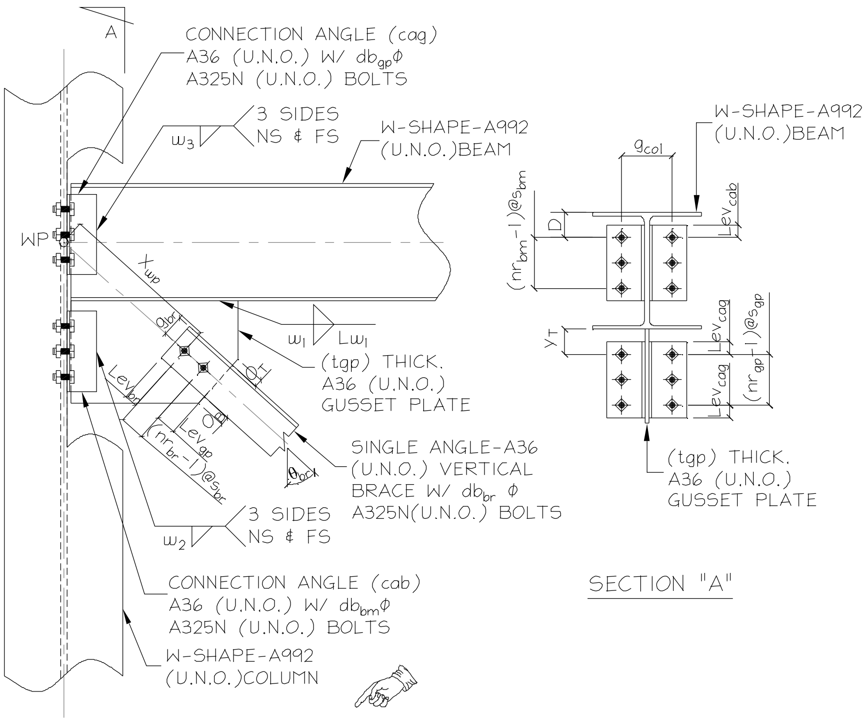
$$Rwbn_g = \Lambda_b \cdot \left(\frac{\psi_1 \cdot L_{yla}}{b_1} + 4 \cdot \psi_1 \cdot \sqrt{1 + \frac{2 \cdot g_{cab} + tw_{bm}}{2 \cdot b_1}} \right)$$

$Rwbn = 83.21$ kips $\quad\quad$ $\left| H_{bm} - Pu_{bm2} \right| = 0$ kips

RESULT = Web Bending Capacity > Force Applied , LCR = 0, OK

III. DETAILS

A. SKETCH



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR THICKNESS OF PLATE, LENGTH AND SIZE OF WELDS)

VB21-143/ DESIGN OF SINGLE ANGLE VERTICAL BRACE TO W-SHAPE COLUMN WEB AND W-SHAPE BEAM CONNECTION

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B. TABLE: VB21-143 / VERTICAL BRACE SCHEDULE

Brace Mark	Vertical Brace	θ_{br} (±2°)	Connection Angle		Bolt Diameter		No.of Bolts		Gusset Plate		Weld Size and Length (in)			Brace Load (kips)
			Size	Grade	db_{br}	db_{gp}	nr_{br}	nr_{gp}	tgp	Grade	w_1	w_2	Lw_1	P_{br}
A3000(?)	L6X4X3/8	53.129	L4X4X5/16	A36	3/4	3/4	4	2	1/4	A36	1/4	3/16	32.337	39

Column Mark	Beam Mark	Column	Beam	Connection Angle		db_{col} (in)	No.of Bolts	D (in)	y_T (in)	Weld Size (in)	g_{col} (in)	Beam Shear Load (kips)	Axial Load (kips)
				Size	Grade		nr_{bm}			w_3			
C1101(?)	B1010(?)	W12X96	W21X44	L4X3-1/2X1/4	A36	3/4	4	3	3	3/16	5 1/2	50	5

REMARKS ON BRACE TO GUSSET CONNECTION			REMARKS ON GUSSET TO BEAM FLANGE CONNECTION
Bolt Spacing	Edge Distance	Bolts	For Weld on Gusset Plate to Beam Flange (w_1)
		Ok, LCR = 0.954	Ok, LCR = 0.322

REMARKS ON GUSSET PLATE TO COLUMN CONNECTION					
Bolt Spacing	Edge Distance	Bolts	Clip Angle	For Weld on Angle to Gusset PLate (w_2)	For Bolt Tension & Elastic Moment Capacity
		Ok, LCR = 0.167		Ok, LCR = 0.161.	Ok, LCR = 0

REMARKS ON BEAM TO COLUMN CONNECTION						
Bolt Spacing	Edge Distance	Bolts	Clip Angle	Angle Length	For Weld on Angle to Beam Web (w_3)	For Bolt Tension Capacity
		Ok, LCR = 0.782	Ok, LCR = 0.971		OK, LCR = 0.698	Ok, LCR = 0

REMARKS ON OTHER CONNECTING ELEMENTS			
Brace Member	Gusset Plate	Column (due to Gusset Stresses & Forces on Beam)	Beam (due to Gusset Stresses & Forces on Beam)
Ok, LCR = 0.537	Ok, LCR = 0.859	Ok, LCR = 0.055	Ok, LCR = 0.345

Note:

1. All Welds are E70XX LH
2. LLBB - Long Leg Back to Back
3. SLBB - Short Leg Back to Back

IV. REFERENCES

IDS Connection Design Standards, 2005

Steel Construction Manual (13th Ed.), American Institute of Steel Construction, Inc., 2005