JOB: TSM1

SC01-11/SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE GIRDER CLIP ANGLE CONNECTION (BOLTED-BOLTED)

I. DESIGN DATA AND LOAD (ASD)

(B1999(?) BEAM TO B1999(?) GIRDER) CONN. ID - 3175997

GIRDER PROPERTIES (gir): W21X44 - A992(B1999(?))

Fy_{gir} = 50 ksi d_{gir} = 20.7 in tw_{gir} = 0.35 in $k1_{gir}$ = 0.812 in

Fu_{gir} = 65 ksi bf_{gir} = 6.5 in tf_{gir} = 0.45 in k_{gir} = 1.125 in

gap = 0.5 in   E:= 29,000 ksi

Girder Top of Steel Elevation, $Elev_1$ = 25'-0"

BEAM PROPERTIES (bm): W16X26 - A992(B1999(?))

Fy_{bm} = 50 ksi d_{bm} = 15.7 in tw_{bm} = 0.25 in $k1_{bm}$ = 0.75 in

Fu_{bm} = 65 ksi bf_{bm} = 5.5 in tf_{bm} = 0.345 in k_{bm} = 1.062 in

Beam Top of Steel Elevation, $Elev_2$ = 25'-0"

Distance of First Bolt from Beam Flange, D = 3.5 in

Length of Beam, L_{bm} = 85.813 in

	TOP COPE	*BOTTOM COPE*
Cope Dimensions,	dc_T = 1 in	dc_B = 0 in
	c_T = 3.25 in	c_B = 0 in

CLIP ANGLE PROPERTIES (ca): L4X3-1/2X1/4 - A36

Fy_{ca} = 36 ksi $leg1_{ca}$ = 4 in   tca = 0.25 in

Fu_{ca} = 58 ksi $leg2_{ca}$ = 3.5 in   Number of Clip Angle: n_{ca}:= 2

Beam side bolt gage: g_{cab} = 2.25 in

Column side bolt gage: g_{cag} = 2.625 in

BOLTS:

db = 0.75 in   Bolt Type = A325N

Arv = 10.603 kips   Hole Diameter:

s = 3 in   Clip angle(Girder side), hd_{gcav} = 0.875 in hd_{gcah} = 1.062 in

Lev = 1.5 in   Clip angle(Beam side), hd_{bcav} = 0.875 in hd_{bcah} = 0.875 in

Leh_{bm} = 1.75 in   Girder, hd_{girv} = 0.875 in hd_{girh} = 0.875 in

Beam, hd_{bmv} = 0.875 in hd_{bmh} = 0.875 in number of bolt rows: $nr_g = 3$

number of vertical bolt lines: $nv = 1$

total number of bolts: $n = nr_g \cdot nv$ $n = 3$

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Resistance Factor, ϕ (LRFD)

Modification Factor, $\Lambda = \dfrac{1}{\Omega}$ (IF ASD) $\Lambda = \phi$ (IF LRFD)

	safety factor	resistance factor	modification factor
For bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.5$
For block shear,	$\Omega_{bs} = 2.00$	$\phi_{bs} = 0.75$	$\Lambda_{bs} = 0.5$
For flexural local buckling,	$\Omega_{b} = 1.67$	$\phi_{b} = 0.90$	$\Lambda_{b} = 0.6$
For flexural rupture,	$\Omega_{fr} = 2.00$	$\phi_{fr} = 0.75$	$\Lambda_{fr} = 0.5$
For shear,	$\Omega_{v} = 1.67$	$\phi_{v} = 0.90$	$\Lambda_{v} = 0.6$
For shear on bolts (Bearing Type),	$\Omega_{vtn} = 2.00$	$\phi_{vtn} = 0.75$	$\Lambda_{vtn} = 0.5$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.5$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 0.67$

APPLIED LOAD:

Shear Load of Beam, $V = 40$ kips (0 % UDL)

II. CALCULATIONS:

A. BOLTS CHECK

1. Check if Given No. of Bolts are Adequate

a. *Bolts on Beam*

Given No. of Bolts,

$nr_g = 3$

Required Number of Bolts,

$$nr1_{req} = \mathrm{Ceil}\left(\frac{V}{n_{ca} \cdot \Lambda rv}, 1\right) \qquad nr1_{req} = 2$$

RESULT = Given Number of Bolts are Adequate

b. *Bolts on Girder*

Required Number of Bolts,

$$nr2_{req} = \mathrm{Ceil}\left(\frac{V}{n_{ca} \cdot \Lambda rv}, 1\right) \qquad nr2_{req} = 2$$

RESULT = Given Number of Bolts are Adequate

c. *Governing No. of Bolts to Design the Connection*

Required Number of Bolts,

$nr_{reqT} = \max(nr1_{req}, nr2_{req})$ $nr_{reqT} = 2$

Governing No of Bolts,

$nr = 3$

Maximum Number of Bolts Applicable in the Connection,

$nr_{max} = 4$

RESULT = Required No of Bolts are OK to Satisfy the Load on Connection

2. Bolt Shear Capacity

a. *Bolts on Beam*

Shear Capacity per Bolt,

$Arv = 10.603$ kips

Bolt Shear Capacity,

$Rb_{v1} = n_{ca} \cdot nr \cdot Arv$

$Rb_{v1} = 63.617$ kips $V = 40$ kips

RESULT = Bolt Shear Capacity > Force Applied, LCR = 0.629, OK

b. *Bolts on Girder*

Shear Capacity per Bolt,

$Arv = 10.603$ kips

Bolt Shear Capacity,

$Rb_{v2} = n_{ca} \cdot nr \cdot Arv$

$Rb_{v2} = 63.617$ kips $V = 40$ kips

RESULT = Bolt Shear Capacity > Force Applied, LCR = 0.629, OK

3. Check for spacing

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

$s = 3$ in

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

$s_{max} = \min(12in, 24 \cdot \min(tw_{bm}, tca, tw_{gir}))$ $s_{max} = 6$ kips

RESULT = s > s_min & s < s_max, OK

4. Check for vertical 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

$Lev_{min} = Le_{min} + C_2$ $Lev_{min} = 1$ in

$Lev_{max} = \min(6in, 12 \cdot \min(tw_{bm}, tca, tw_{gir}))$ $Lev_{max} = 3$ 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)

$Leh_{ca} = \min(leg1_{ca} - g_{cag}, leg2_{ca} - g_{cab})$ $\quad Leh_{ca} = 1.25$ in

$Leh_{min} = Le_{min} + C_2$ $\quad Leh_{min} = 1.125$ in

$Leh_{max} = \min(6in, 12 \cdot \min(tw_{bm}, tca, tw_{gir}))$ $\quad Leh_{max} = 3$ in

RESULT = Leh > Leh_min & Leh < Leh_max, OK

B. BEAM CHECK

1. Bolt Bearing Capacity on Beam

(AISC 13th Ed. Chapter J, section J3.10, pages 16.1-111)

$Abrg_{bm} = db \cdot tw_{bm}$ $\quad Abrg_{bm} = 0.188\ in^2$

Allowable Bearing Strength using edge distance,

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

$Fbe = Fu_{bm} \cdot \min[1.0 \cdot [D-(dc_T+0.5hd_{bmv})] \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}]$

Otherwise,

$Fbe = Fu_{bm} \cdot \min[1.2 \cdot [D-(dc_T+0.5hd_{bmv})] \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}]$

$Fbe = 29.25$ kips

Allowable Bearing Strength using bolt spacing,

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

$Fbs = Fu_{bm} \cdot \min[1.0 \cdot (s - hd_{bmv})] \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}]$

Otherwise,

$Fbs = Fu_{bm} \cdot \min[1.0 \cdot (s - hd_{bmv})] \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}]$

$Fbs = 29.25$ kips

Bolt Bearing Capacity,

$Rbrg_{bm} = Abrg \cdot nv \cdot [Fbe + Fbs \cdot (nr - 1)]$

$Rbrg_{bm} = 43.875$ kips $\quad V = 40$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.912,OK

2. Coped Beam Capacity

a. *Capacity if beam web is single coped at top*

(AISC 13th Ed. Chapter 9, pages 9-6 to 9-7)

Depth of Top Cope,

$dc = dc_T$ $\quad dc = 1$ in

RESULT = depth of cope < 0.5 of depth of beam, OK

Length of Cope,

$c = c_T$ $\qquad$ $c = 3.25$ in

RESULT = length of cope < twice the depth of beam, OK

Reduced Beam Depth

$ho = d_{bm} - dc$ $\qquad$ $ho = 14.7$ in

Plate Buckling Coefficient,

If $\frac{c}{ho} \le 1.0$,

$$k = 2.2 \cdot \left(\frac{ho}{c}\right)^{1.65}$$

Otherwise,

$$k = 2.2 \cdot \frac{ho}{c}$$

$k = 26.539$

Plate Buckling Model Adjustment Factor,

If $\frac{c}{d_{bm}} \le 1.0$,

$$f = 2 \cdot \frac{c}{d_{bm}}$$

Otherwise,

$$f = 1 + \frac{c}{d_{bm}}$$

$f = 0.414$

Allowable Buckling Stress,

$$Fcr = \min\left[26210f \cdot k \cdot \left(\frac{tw_{bm}}{ho}\right)^2 \cdot ksi, Fy_{bm}\right]$$

$Fcr = 50$ ksi

Location of Neutral Axis on the Reduced Section

$do = ho - tf_{bm}$ $\qquad$ $do = 14.355$ in

$$x_b = \frac{do \cdot tw_{bm}\left(\frac{do}{2} + tf_{bm}\right) + bf_{bm} \cdot tf_{bm} \cdot \left(\frac{tf_{bm}}{2}\right)}{do \cdot tw_{bm} + bf_{bm} \cdot tf_{bm}}$$

$x_b = 4.98$ in

$x_t = ho - x_b$ $\qquad$ $x_t = 9.72$ in

Moment of Inertia,

$$I = \frac{tw_{bm} \cdot do^3}{12} + do \cdot tw_{bm} \cdot \left(\frac{do}{2} - x_t\right)^2 + \frac{bf_{bm} \cdot tf_{bm}^3}{12} + bf_{bm} \cdot tf_{bm} \cdot \left(x_b - \frac{tf_{bm}}{2}\right)^2$$

$I = 128.699 \text{ in}^4$

Net Section Modulus,

$$Snet = \frac{I}{\min(x_t, x_b)} \qquad Snet = 13.241 \text{ in}^3$$

Eccentricity,

$e_{bm} = c + gap \qquad e_{bm} = 3.75 \text{ in}$

Buckling Capacity,

$$Rbc = A_b \frac{Fcr \cdot Snet}{e_{bm}} \qquad Rbc = 105.93 \text{ kips}$$

Flexural Rupture Capacity,

$$Rfr = A_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}} \qquad Rfr = 114.757 \text{ kips}$$

Shear Capacity of Reduced Section,

$Vwg_1 = A_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm} \qquad Vwg_1 = 73.504 \text{ kips}$

Coped Beam Capacity,

$Rscb_1 = \min(Rbc, Rfr, Vwg_1)$

$Rscb_1 = 73.504 \text{ kips} \qquad V = 40 \text{ kips}$

RESULT = Coped Beam Capacity > Force Applied, LCR = 0.544,OK

b. *Capacity if beam web is double coped*
(AISC 13th Ed. Chapter 9, page 9-8)

Depth of Cope,

Top Cope: $dc_T = 1$ in

Bottom Cope: $dc_B = 0$ in

Maximum Cope: $dc = \max(dc_T, dc_B) \qquad dc = 1$ in

RESULT = depth of cope < 0.2 of depth of beam, OK

Length of Cope,

Top Cope: $c_T = 3.25$ in

Bottom Cope: $c_B = 0$ in

Maximum Cope: $c = \max(c_T, c_B) \qquad c = 3.25$ in

RESULT = length of cope < twice the depth of beam, OK

Reduced Beam Depth,

$ho = d_{bm} - dc_T - dc_B$ $\quad\quad ho = 14.7\ in$

Adjustment Factor of Lateral-Torsional Buckling Model,

$$fd = 3.5 - 7.5\left(\frac{dc}{d_{bm}}\right) \quad\quad fd = 3.022$$

Allowable Buckling Stress,

$$Fcr = \min\left(0.62\cdot\pi\cdot Efd\cdot\frac{tw_{bm}^{\ 2}}{c\cdot ho}, Fy_{bm}\right) \quad Fcr = 50\ ksi$$

Net Section Modulus,

$$Snet = \frac{tw_{bm}\cdot ho^2}{6} \quad\quad Snet = 9.004 in^3$$

Eccentricity,

$e_{bm} = c + gap$ $\quad\quad e_{bm} = 3.75\ in$

Buckling Capacity,

$$Rbc = \phi_b \frac{Fcr\cdot Snet}{e_{bm}} \quad\quad Rbc = 72.03\ kips$$

Flexural Rupture Capacity,

$$Rfr = \phi_{fr}\frac{Fu_{bm}\cdot Snet}{e_{bm}} \quad\quad Rfr = 78.032\ kips$$

Shear Capacity of Reduced Section,

$$Vwg_2 = \phi_{vy}\cdot 0.6\cdot Fy_{bm}\cdot ho\cdot tw_{bm} \quad\quad Vwg_2 = 73.504\ kips$$

Coped Beam Capacity,

$Rdcb = \min(Rbc, Rfr, Vwg_2)$

$Rdcb = 72.03\ kips$ $\quad\quad V = 40\ kips$

RESULT = Coped Beam Capacity > Force Applied, LCR = 0.555,OK

c. *Deep Coped Beam* *(for dc > 0.2dbm)*
(AISC 13th Ed. Chapter 9, pages 9-8 to 9-9)

Coefficient,

$$\lambda = \frac{ho\cdot\sqrt{Fy_{bm}}}{10\cdot tw_{bm}\cdot\sqrt{475+280\cdot\left(\frac{ho}{c}\right)^2}}\cdot\sqrt{\frac{1}{ksi}}$$

$\lambda = 0.528$

Value of Q,

If $\lambda \le 0.7$,

$Q = 1$

If $0.7 < \lambda \le 1.41$,

$Q = 1.34 - 0.486 \cdot \lambda$

Otherwise,

$$Q = \frac{1.30}{\lambda^2}$$

$Q = 1$

Allowable Buckling Stress,

$Fcr = Fy_{bm} \cdot Q$ $\qquad$ $Fcr = 50$ ksi

Net Section Modulus,

$$Snet = \frac{tw_{bm} \cdot ho^2}{6}$$ $\qquad$ $Snet = 9.004\ in^3$

Eccentricity,

$e_{bm} = c + gap$ $\qquad$ $e_{bm} = 3.75$ in

Buckling Capacity,

$$Rbc = \Phi_b \frac{Fcr \cdot Snet}{e_{bm}}$$ $\qquad$ $Rbc = 72.03$ kips

Flexural Rupture Capacity,

$$Rfr = \Phi_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$ $\qquad$ $Rfr = 78.032$ kips

Shear Capacity of Reduced Section,

$Vwg_3 = \Phi_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$ $\qquad$ $Vwg_3 = 73.504$ kips

Coped Beam Capacity,

$Rtdcb = \min(Rbc, Rfr, Vwg_3)$

$Rtdcb = 72.03$ kips $\qquad$ $V = 40$ kips

RESULT = Coped Beam Capacity > Force Applied, LCR = 0.555,OK

d. *Capacity if beam web is single coped at bottom*

(AISC 13th Ed. Chapter J, Section J.2 and J.4, page 16.1-113)

Depth of Cope,

$dc = dc_B$ $\qquad$ $dc = 0$ in

RESULT = depth of cope < 0.5 of depth of beam, OK

Coped Depth,

$ho = d_{bm} - dc_B$ $\qquad ho = 15.7\ in$

Length of Cope,

$c = c_B$ $\qquad c = 0.01\ in$

RESULT = length of cope < twice the depth of beam, OK

Allowable Flexural Buckling Stress,

$Fbc = \Lambda_b \cdot Fy_{bm}$ $\qquad Fbc = 30\ ksi$

Location of Neutral Axis on the Reduced Section,

$do = ho - tf_{bm}$ $\qquad do = 15.355\ in$

$$x_t = \frac{do \cdot tw_{bm} \left(\frac{do}{2} + tf_{bm}\right) + bf_{bm} \cdot tf_{bm} \cdot \left(\frac{tf_{bm}}{2}\right)}{do \cdot tw_{bm} + bf_{bm} \cdot tf_{bm}}$$

$x_t = 5.426\ in$

$x_b = ho - x_t$ $\qquad x_b = 10.274\ in$

Moment of Inertia,

$$I = \frac{tw_{bm} \cdot do^3}{12} + do \cdot tw_{bm} \cdot \left(x_b - \frac{do}{2}\right)^2 + \frac{bf_{bm} \cdot tf_{bm}^3}{12} + bf_{bm} \cdot tf_{bm} \cdot \left(x_t - \frac{tf_{bm}}{2}\right)^2$$

$I = 153.692\ in^4$

Net Section Modulus,

$Snet = \frac{I}{\min(x_b, x_t)}$ $\qquad Snet = 14.959\ in^3$

Eccentricity,

$e_{bm} = c + gap$ $\qquad e_{bm} = 0.51\ in$

Buckling Capacity,

$Rbc = \frac{Fbc \cdot Snet}{e_{bm}}$ $\qquad Rbc = 879.944\ kips$

Gross Web Shear,

$Vwg_4 = \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$ $\qquad Vwg_4 = 78.504\ kips$

Coped Beam Capacity,

$Rscb_2 = \min(Rbc, Vwg_4)$

$Rscb_2 = 78.504\ kips$ $\qquad V = 40\ kips$

RESULT = Coped Beam Capacity > Force Applied, LCR = 0.51,OK

3. Block Shear Capacity of Beam

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

Reduction Factor, $U_{bs} = 1.00$

Gross Shear Area

$Agv = [s \cdot (nr - 1) + (D - dc_T)] \cdot tw_{bm}$ $\quad Agv = 2.125\ in^2$

Net Tension Area

$Ant = (Leh_{bm} - 0.5 \cdot hd_{bm}) \cdot tw_{bm}$ $\quad Ant = 0.328\ in^2$

Net Shear Area

$Anv = [s \cdot (nr - 1) + (D - dc_T) - (nr - 0.5) \cdot hd_{bm}] \cdot tw_{bm}$

$Anv = 1.578\ in^2$

Block Shear Capacity of Beam

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

$Rbs_{bm} = 41.438\ kips$ $\quad V = 40\ kips$

RESULT = Block Shear Capacity > Force Applied, LCR = 0.965,OK

4. Shear Rupture Capacity of beam

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

Net Shear Area

$Anv = (d_{bm} - dc_T - dc_B - nr \cdot hd_{bm}) \cdot tw_{bm}$

$Anv = 3.019\ in^2$

Block Shear Capacity

$Rvr_{bm} = \phi_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot Anv$

$Rvr_{bm} = 58.866\ kips$ $\quad V = 40\ kips$

RESULT = Shear Rupture Capacity > Force Applied, LCR = 0.68,OK

5. Shear Capacity of beam

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

Clear distance between flanges of beam,

$h = d_{bm} - 2 \cdot k_{bm}$ $\quad h = 13.575\ in$

Limiting ratio,

$h_{tw} = \dfrac{h}{tw_{bm}}$ $\quad h_{tw} = 54.3$

Determine if Intermediate stiffeners are required,

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

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

$a = 0$ in

RESULT = Intermediate Stiffeners NOT REQUIRED

Web plate buckling coefficient,

$kv = 5$

Web shear coefficient,

$Cv = 1$

Shear Capacity of Section,

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

$Rv_{bm} = 70.65$ kips $V = 40$ kips

RESULT = Shear Yielding Capacity > Force Applied, LCR = 0.566,OK

C. CLIP ANGLE CHECK

1. Check if Given Thickness of Angle is Adequate

a. *Required Angle Thickness on Bolt Bearing*

Length of Allowable Bearing Strength using Edge Distance,

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

$x = \min[1.0 \cdot (Lev - 0.5hd_{bcav}), 2.0 \cdot db]$

Otherwise,

$x = \min[1.2 \cdot (Lev - 0.5hd_{bcav}), 2.4 \cdot db]$

$x = 1.275$ in

Length of Allowable Bearing Strength using Spacing,

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

$y = \min[1.0 \cdot (s - hd_{bcav}), 2.0 \cdot db]$

Otherwise,

$y = \min[1.2 \cdot (s - hd_{bcav}), 2.4 \cdot db]$

$y = 1.8$ in

Required Angle of Thickness,

$$tca_{brg} = \frac{V}{\phi_{brg} \cdot n_{ca} \cdot Fu_{ca} \cdot [x + y \cdot (nr - 1)]}$$

$tca_{brg} = 0.141$ in

RESULT = Given Thickness of Angle is OK per Bolt Bearing

b. *Required Angle Thickness on Shear Yielding*

Length of Angle,

$Lca = (nr - 1) \cdot s + 2 \cdot Lev$ $\qquad$ $Lca = 9$ in

Required Angle of Thickness,

$$tca_{vy} = \frac{V}{\phi_{vy} \cdot 0.6 \cdot Fy_{ca} \cdot Lca \cdot n_{ca}} \qquad tca_{vy} = 0.154 \text{ in}$$

RESULT = Given Thickness of Angle is OK per Shear Yielding

c. *Required Angle Thickness on Shear Rupture*

Required Angle Thickness,

$$tca_{vr} = \frac{V}{\phi_{vr} \cdot 0.6 \cdot Fu_{ca} \cdot n_{ca} \cdot (Lca - nr \cdot \max(hd_{bcav}, hd_{gcav}))}$$

$tca_{vr} = 0.18$ in

RESULT = Given Thickness of Angle is OK per Shear Rupture

d. *Required Angle Thickness on Block Shear*

Girder/ Support Side:

Gross Shear Length,

$Agv = n_{ca} \cdot [s \cdot (nr - 1) + Lev]$ $\qquad$ $Agv = 15$ in

Net Tension Length,

$Ant = n_{ca} \cdot (leg1_{ca} - g_{cag} - 0.5 hd_{gcah})$ $\qquad$ $Ant = 1.688$ in

Net Shear Length,

$Anv = n_{ca} \cdot [s \cdot (nr - 1) + Lev - (nr - 0.5) \cdot hd_{gcav}]$

$Anv = 10.625$ in

$x1_{brg} = \min(0.6 Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant, 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot Ant)$

$x1_{brg} = 421.875$ kips / in

Required Angle Thickness on Support Side,

$$tca1_{bs} = \frac{V}{\phi_{bs} \cdot x1_{brg}} \qquad tca1_{bs} = 0.19 \text{ in}$$

Beam Side:

Gross Shear Length,

$Agv = n_{ca} \cdot [s \cdot (nr - 1) + Lev]$ $\quad Agv = 15$ in

Net Tension Length,

$Ant = n_{ca} \cdot (leg2_{ca} - g_{cab} - 0.5hd_{bcah})$ $\quad Ant = 1.625$ in

Net Shear Length,

$Anv = n_{ca} \cdot [s \cdot (nr - 1) + Lev - (nr - 0.5) \cdot hd_{bcav}]$

$Anv = 10.625$ in

$x2_{brg} = \min(0.6Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant, 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot Ant)$

$x2_{brg} = 418.25$ kips / in

Required Angle Thickness on Beam Side,

$$tca2_{bs} = \frac{V}{\Lambda_{bs} \cdot x2_{brg}} \qquad tca2_{bs} = 0.191 \text{ in}$$

Governing Angle Thickness per Block Shear,

$tca_{bs} = \max(tca1_{bs}, tca2_{bs})$ $\quad tca_{bs} = 0.191$ in

RESULT = Given Thickness of Angle is OK per Block Shear

e. *Governing Angle Thickness to be used*

Required Thickness of Angle to Design the Connection,

$tca_{req} = \max(tca_{brg}, tca_{vy}, tca_{vr}, tca_{bs})$

$tca_{req} = 0.191$ in

RESULT = Required Angle Thickness is OK with Angle Size

Governing Thickness of Angle,

$tca_{eq} = 0.25$ in

2. Bolt Bearing Capacity on Clip Angle

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

$Abrg_{ca} = db \cdot tca_{eq}$ $\quad Abrg_{ca} = 0.188 \text{ in}^2$

Allowable Bearing Strength using edge distance,

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

$Fbe = Fu_{ca} \cdot \min[1.0 \cdot (Lev - 0.5hd_{bcav}) \cdot tca_{eq}, 2.0 \cdot Abrg_{ca}]$

Otherwise,

$Fbe = Fu_{ca} \cdot \min[1.2 \cdot (Lev - 0.5hd_{bcav}) \cdot tca_{eq}, 2.4 \cdot Abrg_{ca}]$

$Fbe = 18.487$ kips

Allowable Bearing Strength using bolt spacing,

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

$$Fbs = Fu_{ca} \cdot \min[1.0 \cdot (s - hd_{bcav}) \cdot tca_{eq}, 2.0 \cdot Abrg_{ca}]$$

Otherwise,

$$Fbs = Fu_{ca} \cdot \min[1.0 \cdot (s - hd_{bcav}) \cdot tca_{eq}, 2.0 \cdot Abrg_{ca}]$$

$Fbs = 26.1$ kips

Bolt Bearing Capacity

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

$Rbrg_{ca} = 70.688$ kips $V = 40$ kips

RESULT = Bearing Capacity > Force Applied, LCR = 0.566,OK

3. Shear Yielding Capacity on Clip Angle

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

Length of Angle,

$Lca = 9$ in

Check if Length of Angle is acceptable per AISC requirements,

If $Lca \geq 0.5(d_{bm} - 2k_{bm})$,

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,

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

$Rvy_{ca} = 64.803$ kips $V = 40$ kips

RESULT = Shear Yielding Capacity > Force Applied, LCR = 0.617,OK

4. Shear Rupture Capacity of Clip Angle

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

Net Shear Area,

$$Anv = (Lca - nr \cdot \max(hd_{bcav}, hd_{gcav})) \cdot tca_{eq}$$

$Anv = 1.594\ in^2$

Shear Rupture Capacity,

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

$Rvr_{ca} = 55.462$ kips $V = 40$ kips

RESULT = Shear Rupture Capacity > Force Applied, LCR = 0.721,OK

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5. Block Shear Capacity of Clip Angle

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

Reduction Factor, $U_{bs} = 1.00$

Girder/ Support Side:

Gross Shear Area,

$Agv = n_{ca} \cdot [s \cdot (nr - 1) + Lev] \cdot tca_{eq}$ $Agv = 3.75\ in^2$

Net Tension Area,

$Ant = n_{ca} \cdot (leg1_{ca} - g_{cag} - 0.5hd_{gcah}) \cdot tca_{eq}$

$Ant = 0.422\ in^2$

Net Shear Area,

$Anv = n_{ca} \cdot [s \cdot (nr - 1) + Lev - (nr - 0.5) \cdot hd_{gcav}] \cdot tca_{eq}$

$Anv = 2.656\ in^2$

Block Shear Capacity on Girder Side,

$Rbs_{ca1} = \phi_{bs} \cdot min(0.6 \cdot Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant, 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot$

$Rbs_{ca1} = 52.734\ kips$

Beam Side:

Reduction Factor, $U_{bs} = 1.00$

Gross Shear Area,

$Agv = n_{ca} \cdot [s \cdot (nr - 1) + Lev] \cdot tca_{eq}$ $Agv = 3.75\ in^2$

Net Tension Area,

$Ant = n_{ca} \cdot (leg2_{ca} - g_{cab} - 0.5hd_{bcah}) \cdot tca_{eq}$

$Ant = 0.406\ in^2$

Net Shear Area,

$Anv = n_{ca} \cdot [s \cdot (nr - 1) + Lev - (nr - 0.5) \cdot hd_{bcav}] \cdot tca_{eq}$

$Anv = 2.656\ in^2$

Block Shear Capacity on Beam Side,

$Rbs_{ca2} = \phi_{bs} \cdot min(0.6 \cdot Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant, 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot$

$Rbs_{ca2} = 52.281\ kips$

Block Shear Capacity on Clip Angle,

$Rbs_{ca} = min(Rbs_{ca1}, Rbs_{ca2})$

$Rbs_{ca} = 52.281\ kips$ $V = 40\ kips$

RESULT = Block Shear Capacity > Force Applied, LCR = 0.765,OK

D. GIRDER CHECK

1. Bolt Bearing capacity on Girder

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

Effective Web Thickness for Bearing,

$$tw_{eff} = tw_{gir} \cdot \left(\frac{V}{V + V_{bm2}}\right) \qquad tw_{eff} = 0.35 \text{ in}$$

Allowable Bearing Strength using edge distance,

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

$$Fbe = Fu_{gir} \cdot tw_{eff} \cdot \min[1.0 \cdot [d_{gir} - (nr - 1)s - D - 0.5hd_{girv}], 2.0 \cdot db]$$

Otherwise,

$$Fbe = Fu_{gir} \cdot tw_{eff} \cdot \min[1.2 \cdot [d_{gir} - (nr - 1)s - D - 0.5hd_{girv}], 2.4 \cdot db]$$

$$Fbe = 40.95 \text{ kips}$$

Allowable Bearing Strength using bolt spacing,

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

$$Fbs = Fu_{gir} \cdot \min[1.0 \cdot (s - hd_{girv}) \cdot tw_{eff}, 2.0 \cdot (db \cdot tw_{eff})]$$

Otherwise,

$$Fbs = Fu_{gir} \cdot \min[1.2 \cdot (s - hd_{girv}) \cdot tw_{eff}, 2.4 \cdot (db \cdot tw_{eff})]$$

$$Fbs = 40.95 \text{ kips}$$

Bolt Bearing Capacity,

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

$$Rbrg_{gir} = 122.85 \text{ kips} \qquad V = 40 \text{ kips}$$

RESULT = Bearing Capacity > Force Applied, LCR = 0.326,OK

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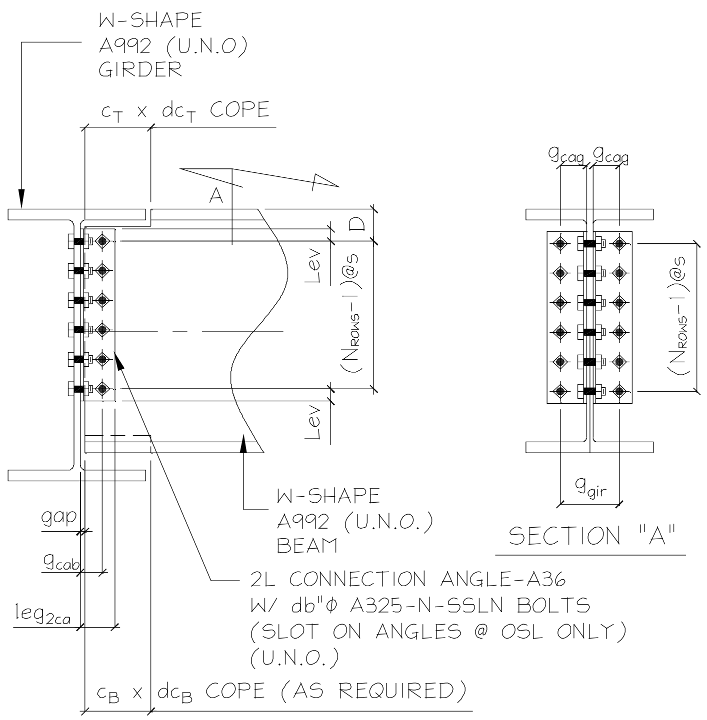
III. DETAILS

A. SKETCH



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED SHEAR CONNECTION SCHEDULE FOR NUMBER OF BOLTS, COPE DEPTH, AND COPE LENGTH.

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Prepared By: LCO Date: 09/01/2009
Checked By: ABS Date: 09/15/2009
Subject: SC01-11

B. W-SHAPE BEAM TO W-SHAPE GIRDER CLIP ANGLE CONNECTION (BOLTED- BOLTED) SCHEDULE

Girder Mark	Girder		Beam		D	Cope Dimensions (in)				g_{gir}	gap	V	Rcap	Governing Capacity
	Size	Grade	Size	Grade	(in)	dc_T	c_T	dc_B	c_B	(in)	(in)			
B1999(?)	W21X44	A992	W16X26	A992	3 1/2	1	3 1/4	0	0	5 1/2	1/2	40	41.438	Block Shear of Beam

Beam Mark	Connection Angle					Bolt Type	Bolts		s	Lev	Remarks for Connecting Members
	Size	Grade	g_{cab}	g_{cag}	$leg2_{ca}$		db (in)	N_{ROWS}	(in)	(in)	
B1999(?)	L4X3-1/2X1/4	A36	2 1/4	2 5/8	3 1/2	A325N	3/4	3	3	1 1/2	

Governing Load Capacity Ratio	Recommendation
0.965	

Remarks on Connection/ Connecting Elements				
For Bolts	For Angle	For Bolt Spacing	For Angle Edge Distance	For Angle Length
Ok, LCR = 0.629	Ok, LCR = 0.765			

IV. REFERENCES

IDS Connection Design Standards, 2005

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

Revision No.	Revision Date	Revised By	Description
00	04/27/2010	GPT	Test Run