SHEAR CONNECTION: W BEAM WITH DOUBLE ANGLE (BOLTED/BOLTED)
ONE-WAY SHEAR CONNECTION TO W GIRDER WEB
I. DESIGN DATA AND LOADS (ASD-14th Edition)

**GIRDER PROPERTIES: W12X40 - A992**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth, d</td>
<td>11.9 in</td>
</tr>
<tr>
<td>Flange Width, bf</td>
<td>8.01 in</td>
</tr>
<tr>
<td>Distance k, k</td>
<td>1.375 in</td>
</tr>
<tr>
<td>Area, Ag</td>
<td>11.7 in²</td>
</tr>
<tr>
<td>Minimum Yield Stress, Fy</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Modulus of Elasticity, E</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Top of Steel Elevation, Elev</td>
<td>9 ft + 10 in</td>
</tr>
</tbody>
</table>

**BEAM PROPERTIES: W16X26 - A992**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth, d</td>
<td>15.7 in</td>
</tr>
<tr>
<td>Flange Width, bf</td>
<td>5.5 in</td>
</tr>
<tr>
<td>Distance k, k</td>
<td>1.063 in</td>
</tr>
<tr>
<td>Area, Ag</td>
<td>7.68 in²</td>
</tr>
<tr>
<td>Minimum Yield Stress, Fy</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Modulus of Elasticity, E</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>Top of Steel Elevation, Elev</td>
<td>10 ft + 0 in</td>
</tr>
</tbody>
</table>

**CONNECTION ANGLE PROPERTIES: 2L4X3-1/2X5/16 SLBB - A36**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Side Leg Size, leg1</td>
<td>4 in</td>
</tr>
<tr>
<td>Beam Side Leg Size, leg2</td>
<td>3.5 in</td>
</tr>
<tr>
<td>Minimum Yield Stress, Fy</td>
<td>36 ksi</td>
</tr>
<tr>
<td>Girder Side Leg Size, t</td>
<td>0.313 in</td>
</tr>
<tr>
<td>Number of Connection Angles, n</td>
<td>2</td>
</tr>
<tr>
<td>Modulus of Elasticity, E</td>
<td>29000 ksi</td>
</tr>
</tbody>
</table>
Minimum Tensile Stress, \( F_u = 58 \text{ ksi} \)

Girder Side Bolt Gage, \( g_1 = 2.5 \text{ in} \)

Beam Side Bolt Gage, \( g_2 = 2.25 \text{ in} \)

**HORIZONTAL STIFFENER PLATE PROPERTIES:** A572-50

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, ( t )</td>
<td>0.375 in</td>
</tr>
<tr>
<td>Width, ( b )</td>
<td>2.625 in</td>
</tr>
<tr>
<td>Minimum Yield Stress, ( F_y )</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Modulus of Elasticity, ( E )</td>
<td>29000 ksi</td>
</tr>
</tbody>
</table>

**BOLTS PROPERTIES:** 3/4" - ø - A325-N

**For Connection Angle to Beam Web Connection:**

- Bolt Diameter, \( d_b = 0.75 \text{ in} \)
- Bolt Shear Strength, \( \Lambda_{rv} = 11.928 \text{ kips} \)
- Bolt Type, \( \text{Bolt}_{-} \text{Type} = \text{A325-N} \)
- Number of Bolt Rows, \( n_r = 3 \)
- Number of Bolt Column Lines, \( n_v = 1 \)
- Total Number of Bolts \( (n_r \cdot n_v) \), \( n_b = 3 \)
- Holes at Beam Web, \( h_{dv} = 0.875 \text{ in} \)
- Vertical Hole Dimension, \( \text{Lev} = 2.438 \text{ in} \)
- Horizontal Hole Dimension, \( \text{Leh} = 1.75 \text{ in} \)
- Bolt First Down from Top of Beam, \( D = 5.438 \text{ in} \)
- Vertical Edge Distance (D-dcT), \( \text{Leg} = 2.438 \text{ in} \)
- Horizontal Edge Distance, \( \text{Leg} = 1.75 \text{ in} \)

- Bolt Tensile Strength, \( \Lambda_{rn} = 19.88 \text{ kips} \)
- Connection Type, \( \text{Conn}_{-} \text{type} = \text{Bearing Type} \)
- Bolt Vertical Spacing, \( s = 3 \text{ in} \)
- Bolt Horizontal Spacing, \( sv = 0 \text{ in} \)

**BOLTS PROPERTIES:** 3/4" - ø - A325-N
For Connection Angle to Girder Web Connection:

Bolt Diameter, \( db = 0.75 \text{ in} \)

Bolt Shear Strength, \( \Lambda_{rv} = 11.928 \text{ kips} \)

Bolt Type, \( \text{Bolt Type} = \text{A325-N} \)

Number of Bolt Rows, \( nr = 3 \)

Number of Bolt Column Lines, \( nv = 1 \)

Total Number of Bolts \( (nr \cdot nv), nb = 3 \)

Adjacent Number of Bolt Rows (if any), \( nr_2 = 0 \)

For Horizontal Stiffener Plate to Beam Web Connection (As Req'd):

Preferred Weld Size \( w = 0.25 \text{ in} \)

WELDS PROPERTIES: \( \text{E70xx LH} \)

Minimum Tensile Stress, \( Fu = 70 \text{ ksi} \)

SAFETY AND RESISTANCE FACTORS:

Safety Factor, \( \Omega(ASD) \)  \quad \text{Resistance Factor,} \( \phi(LRFD) \)

Modification Factor, \( \Lambda = \frac{1}{\Omega} \quad \text{(if ASD)} \)

\( \Lambda = \phi \quad \text{(if LRFD)} \)

\( \text{safety factor} \quad \text{resistance factor} \quad \text{modification factor} \)
For Member in Bearing/Bolt Bearing (brg),
  \( \Omega_{brg} = 2.00 \quad \phi_{brg} = 0.75 \quad \Lambda_{brg} = 0.50 \)

For Block Shear (bs),
  \( \Omega_{bs} = 2.00 \quad \phi_{bs} = 0.75 \quad \Lambda_{bs} = 0.50 \)

For Fillet Weld Shear (vw),
  \( \Omega_{vw} = 2.00 \quad \phi_{vw} = 0.75 \quad \Lambda_{vw} = 0.50 \)

For Flexural Local Buckling/Flexural Strength (b),
  \( \Omega_{b} = 1.67 \quad \phi_{b} = 0.90 \quad \Lambda_{b} = 0.60 \)

For Flexural Rupture (fr),
  \( \Omega_{fr} = 2.00 \quad \phi_{fr} = 0.75 \quad \Lambda_{fr} = 0.50 \)

For Member Shear for C, WT, L (v),
  \( \Omega_{v} = 1.67 \quad \phi_{v} = 0.90 \quad \Lambda_{v} = 0.60 \)

For Shear Rupture (vr),
  \( \Omega_{vr} = 2.00 \quad \phi_{vr} = 0.75 \quad \Lambda_{vr} = 0.50 \)

For Shear Yielding (vy),
  \( \Omega_{vy} = 1.50 \quad \phi_{vy} = 1.00 \quad \Lambda_{vy} = 0.67 \)

APPLIED LOADS:

**Beam:**

50% Uniform Distributed Load

Shear Load, \( V = 29.5 \) kips

Adjacent Shear Load (if any), \( V_2 = 0 \) kips
II. CALCULATIONS

A. BEAM WEB CHECK

1. Bolt Capacity


a. Bolt Capacity due to Shear Load

Bearing Area,
\[ \text{Ab} = \text{db} \times \text{tw} \]
\[ \text{Ab} = 0.187 \text{ in}^2 \]

Bolt Centerline Distance from Face of Support,
\[ \text{ab} = g^2 + 0.5 \times (n - 1) \times sv \]
\[ \text{ab} = 2.25 \text{ in} \]

Eccentricity Distance of End Reaction from Bolt Line,
\[ \text{eb} \leq 3.0 \text{ in} \]
\[ \text{eb} = 0 \text{ in} \]

Load Inclination from Vertical,
\[ \theta = 0 \text{ deg} \]

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)
\[ C = 3 \]

Allowable Bearing Strength Using Edge Distance, (J3-6a, J3-6c)
\[ F_{be} = \Lambda_{brg} \times F_u \times \min \left[ rac{1.2 \times (L_e - 0.5 \times h_d v) \times tw}{2.4 \times Ab_{rg}}, \frac{1.2 \times (L_e - 0.5 \times h_d h) \times tw}{2.4 \times Ab_{rg}} \right] \]
\[ F_{be} = 14.625 \text{ kips} \]

Fbd = \min(Fbe_0, Fbe_2)

Allowable Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)
\[ h_d h < h_d l s \]
\[ F_{bs} = \Lambda_{brg} \times F_u \times \min \left[ 1.2 \times (s - h_d v) \times tw, 2.4 \times Ab_{rg} \right] \]
\[ F_{bs} = 14.625 \text{ kips} \]

Number of Areas in Consideration,
\[ n_1 = 1 \]

Connection Angle,
\[ n_2 = n \]

Bolt Capacity,
\[ \text{eb} \leq 0 \text{ in} \]
R_{brg} = n_v \cdot \left[ \min(n_1 \cdot F_{be}, n_2 \cdot \Lambda_{rv}) + \min(n_1 \cdot F_{bs}, n_2 \cdot \Lambda_{rv}) \cdot (n_r - 1) \right]

R_{brg} = 43.875 \text{ kips} \quad V = 29.5 \text{ kips}

**Bolt Capacity > Applied Force, UCV = 0.672, OK**

2. **Coped Beam Capacity**

   a. **Capacity if Beam Web is Double Coped with Same Cope length at Both Flanges**


   **Depth of Cope,**
   - Top Cope,
     \[ d_{cT} = 3 \text{ in} \]
   - Bottom Cope,
     \[ d_{cB} = 2.75 \text{ in} \]
   - Maximum Cope,
     \[ dc = \max(d_{cT}, d_{cB}) \quad dc = 3 \text{ in} \]

   **depth of cope < 0.2 of depth of beam, OK**

   **Length of Cope,**
   - Top Cope,
     \[ c_{T} = 4 \text{ in} \]
   - Bottom Cope,
     \[ c_{B} = 4 \text{ in} \]
   - Maximum Cope,
     \[ c = \max(c_{T}, c_{B}) \quad c = 4 \text{ in} \]

   **length of cope < twice the depth of beam, OK**

   **Reduced Beam Depth,**
   \[ h_{o} = d - d_{cT} - d_{cB} \]
   \[ h_{o} = 9.95 \text{ in} \]

   **Adjustment Factor of Lateral-Torsional Buckling Model,**
   \[ f_{d} = 3.5 - 7.5 \left( \frac{d_{cT}}{d} \right) \quad f_{d} = 2.067 \]

   **Allowable Flexural Local Buckling Stress or Yielding Stress,**
   \[ F_{cr} = \min \left( 0.62 \cdot \pi \cdot E \cdot f_{d} \cdot \frac{t_{w}^2}{c \cdot h_{o}}, F_{y} \right) \quad F_{cr} = 50 \text{ ksi} \]

   **Net Section Modulus,**
   \[ S_{net} = \frac{t_{w} \cdot h_{o}^2}{6} \quad S_{net} = 4.125 \text{ in}^3 \]

   **Eccentricity,**
\( e = c + \text{gap} \quad e = 4.5 \text{ in} \)

**Flexural Local Buckling Capacity or Yielding Capacity,**

\[
R_{bc} = \Lambda_b \cdot \frac{F_{cr} \cdot S_{net}}{e}
\]

\( R_{bc} = 27.446 \text{ kips} \)

**Flexural Rupture Capacity,**

\[
R_{fr} = \Lambda_{fr} \cdot \frac{F_{u} \cdot S_{net}}{e}
\]

\( R_{fr} = 29.792 \text{ kips} \)

**Shear Capacity of Reduced Section,**

\[
V_{wg} = \Lambda_{vy} \cdot 0.6 \cdot F_{y} \cdot h_0 \cdot t_w
\]

\( V_{wg} = 49.75 \text{ kips} \)

**Coped Beam Capacity,**

\[
R_{cb} = \min (R_{bc}, R_{fr}, V_{wg})
\]

\( R_{cb} = 27.446 \text{ kips} \quad V = 29.5 \text{ kips} \)

Please refer to Design of Coped Beam with Reinforcement, OK

### 3. Coped Beam Capacity with Horizontal Stiffener Plate

**Allowable Flexural Local Buckling Stress/Yielding Stress,**

\( F_{cr} = 50 \text{ ksi} \)

**Location of Neutral Axis on the Reduced Section,**

\( x_b = 6.952 \text{ in} \)
\( x_t = 2.998 \text{ in} \)

**Moment of Inertia,**

\[
I = 42.553 \text{ in}^4
\]

**Slenderness of Horizontal Stiffener Plate,**

**Net Section Modulus at Compression Area,**

\[
S_{xC} = \frac{I}{x_t}
\]

\( S_{xC} = 14.194 \text{ in}^3 \)

**Net Section Modulus at Tension Area,**

\[
S_{xT} = \frac{I}{x_b}
\]

\( S_{xT} = 6.121 \text{ in}^3 \)

**Flexural Yield Stress, (Table B4.1b)**

\[
\frac{S_{xT}}{S_{xC}} < 0.7
\]

\[
FL = \frac{S_{xT}}{S_{xC}} \cdot F_{yl}
\]

\( FL = 21.562 \text{ ksi} \)

**Clear Distance Between Flanges of Beam Less the Fillet or Corner Radii,**

\( h = h_0 \quad h = 9.95 \text{ in} \)

**Web Plate Buckling Coefficient,**
\[
\frac{4}{(\frac{h}{tw})^{0.5}} > 0.35
\]

\[
kc = \min \left( 0.76, \frac{4}{(\frac{h}{tw})^{0.5}} \right)
\]

Limiting Slenderness Parameter for Noncompact Element,

\[
\lambda_r = 0.95 \left( \frac{kc \cdot E}{FL} \right)
\]

\[
\frac{b_{st}}{2t_{st}} \leq \lambda_r
\]

\[
\frac{b_{st}}{2t_{st}} = 3.5
\]

Stiffener Plate is not slender

Net Section Modulus,

\[
S_{net} = \frac{I}{\max(x_{b},x_{t})}
\]

Net Section Modulus,

\[
S_{net} = 6.121 \text{ in}^3
\]

Eccentricity,

\[
e = c + \text{gap} \quad e = 4.5 \text{ in}
\]

Flexural Local Buckling Capacity or Yielding Capacity

\[
R_{bc} = \frac{Ab \cdot F_{cr} \cdot S_{net}}{e}
\]

\[
R_{bc} = 40.725 \text{ kips}
\]

Flexural Rupture Capacity,

\[
R_{fr} = \frac{A_{f} \cdot F_{u} \cdot S_{net}}{e}
\]

\[
R_{fr} = 44.207 \text{ kips}
\]

Shear Capacity of Reduced Section,

\[
V_{wg} = \frac{A_{vy} \cdot 0.6 \cdot F_{y} \cdot h_{o} \cdot tw}{e}
\]

\[
V_{wg} = 49.75 \text{ kips}
\]

Coped Beam Capacity,

\[
R_{cb} = \min(R_{bc}, R_{fr}, V_{wg})
\]

\[
R_{cb} = 40.725 \text{ kips}
\]

\[
V = 29.5 \text{ kips}
\]

Flexural Cope Buckling Capacity > Applied Force, \( UCV = 0.724 \), OK

4. Weld Capacity of Horizontal Stiffener Plate to Beam Web


(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

Number of Weld Sides,

\[
n_{ws} = 4
\]
Minimum Weld Size,
\[ w_{\text{min}} = 0.125 \text{ in} \quad \quad w = 0.25 \text{ in} \]

**Preferred Weld Size > Minimum Weld Size, OK**

Length of Horizontal Stiffener Plate,
\[ L = 7 \text{ in} \]

Force Acting on the Connection,
\[ R_u = 23.865 \text{ kips} \]

Shear Strength,

For Beam Web,
\[ R_v1 = Avr \cdot 0.6 \cdot F_u \cdot t_w \quad \quad R_v1 = 9.75 \text{ kips/in} \]

For Horizontal Stiffener Plate,
\[ R_v2 = Avr \cdot 0.6 \cdot F_u \cdot t \quad \quad R_v2 = 14.625 \text{ kips/in} \]

For Weld,
\[ R_v3 = Awv \cdot 0.6 \cdot F_u \cdot \sin(45\text{deg}) \cdot nws \quad \quad R_v3 = 59.397 \text{ ksl} \]

Maximum Effective Weld Size,
\[ \text{weff} = \min(R_v1, R_v2) \div R_v3 \quad \quad \text{weff} = 0.164 \text{ in} \]

Length of Weld,
\[ L_w = L \quad \quad L_w = 7 \text{ in} \]

Weld Capacity,
\[ R_w = Awv \cdot 0.6 \cdot F_u \cdot \sin(45\text{deg}) \cdot nws \cdot L_w \cdot \min(w, \text{weff}) \quad \quad R_w = 68.25 \text{ kips} \]
\[ R_u = 23.865 \text{ kips} \]

Weld Capacity > Applied Force, UCV = 0.35, OK

5. **Shear Capacity**


Clear Distance Between Flanges of Beam Less the Fillet or Corner Radii,
\[ h = d - 2 \cdot k_{des} \quad \quad h = 14.206 \text{ in} \]

Limiting Depth-thickness Ratio,
\[ h_{tw} = \frac{h}{t_w} \quad \quad h_{tw} = 56.824 \]

Clear Distance Between Transverse Stiffeners,
\[ h_{tw} < 260 \quad \quad a = 0 \text{ in} \]

Web Plate Buckling Coefficient, (G2-6)
\[ h_{tw} < 260 \]
kv = 5
kv = 5

Web Shear Coefficient, (G2-3, G2-4, G2-5)

$$htw \leq 1.1 \left( \frac{kv \cdot E}{Fy} \right)^{0.5}$$

Cv = 1

Shear Capacity, (G2-1)

$$Rv = \lambda_vb \cdot 0.6 \cdot Fy \cdot d \cdot tw \cdot Cv$$

$$Rv = 70,509 \text{ kips} \quad V = 29.5 \text{ kips}$$

Shear Capacity of Section > Applied Force, UCV = 0.418, OK

B. BEAM WEB TO CONNECTION ANGLE CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.6, pages 16.1-125)

Shear Capacity Per Bolt,

$$Arv = 11,928 \text{ kips}$$

Bolt Shear Capacity,

$$Rb = n \cdot nb \cdot Arv$$

$$Rb = 71,569 \text{ kips} \quad V = 29.5 \text{ kips}$$

Bolt Shear Capacity > Applied Force, UCV = 0.412, OK

2. Check for Spacing

(AISC 14th Ed. Specifications Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Connection Angle Thickness,

$$t1 = 0.313 \text{ in}$$

Beam Web Thickness,

$$t2 = 0.25 \text{ in}$$

Vertical Spacing of Bolts,

$$s = 3 \text{ in}$$

$$smin = 2 \cdot \frac{2}{3} \cdot db$$

$$smin = 2 \text{ in}$$

$$smax = \min(12\text{in}, 24 \cdot \min(t1, t2))$$

$$smax = 6 \text{ in}$$

Spacing > Min. Spacing & Spacing < Max. Spacing, OK

3. Check for Edge Distance

(AISC 14th Ed. Specifications, Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Connection Angle Thickness,
t1 = 0.313 in

Connection Angle Edge Distances,
Lev1 = 1.25 in
Leh1 = 1.25 in

Beam Web Thickness,
t2 = 0.25 in

Beam Web Edge Distances,
Lev2 = 2.438 in
Leh2 = NA

Vertical Edge Distance,
Levcon = \left( \begin{array}{c} \text{Lev1} \\ \text{Lev2} \end{array} \right)
Levcon = \left( \begin{array}{c} 1.25 \text{ in} \\ 2.438 \text{ in} \end{array} \right)

Levmin = \left( \begin{array}{c} \text{Levmin1} \\ \text{Levmin2} \end{array} \right)
Levmin = \left( \begin{array}{c} 1 \text{ in} \\ 1 \text{ in} \end{array} \right)

\text{min}(\text{Levcon}) = \text{Lev1}
\text{Levmax} = \text{min}(6\text{ in}, 12 \cdot t1)
\text{Levmax} = 3.756 \text{ in}

\text{Edge Distance} \geq \text{Min. Edge Distance} & \text{Edge Distance} \leq \text{Max. Edge Distance}, \text{ OK}

Horizontal Edge Distance,
Lehcon = \left( \begin{array}{c} \text{Leh1} \\ \text{Leh2} \end{array} \right)
Lehcon = \left( \begin{array}{c} 1.25 \text{ in} \\ \text{NA} \end{array} \right)

Lehmin = \left( \begin{array}{c} \text{Lehmin1} \\ \text{Lehmin2} \end{array} \right)
Lehmin = \left( \begin{array}{c} 1 \text{ in} \\ \text{NA} \end{array} \right)

\text{min}(\text{Lehcon}) = \text{Leh1}
\text{Lehmax} = \text{min}(6\text{ in}, 12 \cdot t1)
\text{Lehmax} = 3.756 \text{ in}

\text{Edge Distance} \geq \text{Min. Edge Distance} & \text{Edge Distance} \leq \text{Max. Edge Distance}, \text{ OK}

C. CONNECTION ANGLE CHECK

1. Bolt Capacity


a. Bolt Capacity due to Shear Load (Secondary Side)

Bearing Area,
\text{Abrg} = \text{db} \cdot \text{t}
\text{Abrg} = 0.235 \text{ in}^2

Bolt Centerline Distance from Face of Support,
\text{ab} = g2 + 0.5 \cdot (nv - 1) \cdot sv
\text{ab} = 2.25 \text{ in}
Eccentricity Distance of End Reaction from Bolt Line,
\[ ab \leq 3.0 \text{ in} \land \text{nv} = 1 \]
\[ ebv = 0 \text{ in} \]

Load Inclination from Vertical,
\[ \theta = 0 \text{ deg} \]

Eccentric Load Coefficient,
\[ C = 3 \]

Allowable Bearing Strength Using Edge Distance, (J3-6a, J3-6c)
\[ hb < hds \]
\[ F_{be} = A_{brg} \cdot F_u \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot A_{brg} \end{bmatrix} \]
\[ ebv \leq 0 \text{ in} \]
\[ F_{be} = \min(F_{be_0}, F_{be_2}) \]
\[ F_{be} = 8.85 \text{ kips} \]

Allowable Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)
\[ hbd < hds \]
\[ F_{bs} = A_{brg} \cdot F_u \cdot \min[1.2 \cdot (s - hdv) \cdot t, 2.4 \cdot A_{brg}] \]
\[ F_{bs} = 16.339 \text{ kips} \]

Number of Areas in Consideration,
\[ n_1 = n \]
\[ n_2 = n \]

Connection Angle,
\[ \theta = \theta \]

Bolt Capacity,
\[ ebv \leq 0 \text{ in} \]
\[ R_{brg1} = \text{nv} \cdot [\min(n_1 \cdot F_{be}, n_2 \cdot A_{rv}) + \min(n_1 \cdot F_{bs}, n_2 \cdot A_{rv}) \cdot (nr - 1)] \]
\[ R_{brg1} = 65.413 \text{ kips} \]
\[ V = 29.5 \text{ kips} \]

Bolt Capacity > Applied Force, UCV = 0.451, OK

b. Bolt Capacity due to Shear Load (Primary Side)

Bearing Area,
\[ A_{brg} = d_b \cdot t \]
\[ A_{brg} = 0.235 \text{ in}^2 \]

Allowable Bearing Strength Using Edge Distance, (J3-6a, J3-6c)
\[ hb < hds \]
\[ F_{be} = A_{brg} \cdot F_u \cdot \min[1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t, 2.4 \cdot A_{brg}] \]
\[ F_{be} = 8.85 \text{ kips} \]

Allowable Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)
Fbs = Abrg·Fu·min[1.2·(s - hdv)·t, 2.4·Abrg]

Fbs = 16.339 kips

Number of Areas in Consideration,

n1 = n

Connection Angle,

n2 = n

Bolt Capacity,

Rbrg2 = n·v·[min(n1·Fbe, n2·Abrg) + min(n1·Fbs, n2·Abrg)·(nr - 1)]

Rbrg2 = 65.413 kips

Bolt Capacity > Applied Force, UCV = 0.451, OK

Governing Bolt Capacity,

Rbrg = min(Rbrg1, Rbrg2)

Rbrg = 65.413 kips

Bolt Capacity > Applied Force, UCV = 0.451, OK

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

L = (nr - 1)·s + 2·Lev

L = 8.5 in

Erection Stability,


Length of Connector > One-half of T-Dimension, OK

Number of Areas in Consideration,

n1 = n

Shear Yielding Capacity, (J4-3)

Rvy = Avy·n1·0.6·Fy·L·t

Rvy = 76.622 kips

Shear Yielding Capacity > Applied Force, UCV = 0.385, OK

3. Rupture Capacity

(AISC 14th Ed. Specifications Chapter J, Section J4.2, page 16.1-129)

a. Shear Rupture Capacity due to Shear Load (Secondary Side)
Net Shear Area,
\[ \text{Anv} = (L - nr \cdot hdv) \cdot t \]
Number of Areas in Consideration,
\[ n_1 = n \]
Shear Rupture Capacity, (J4-4)
\[ \text{Rvr}_1 = \Lambda \cdot v_r \cdot n_1 \cdot 0.6 \cdot F_u \cdot \text{Anv} \]
\[ \text{Rvr}_1 = 63.993 \text{ kips} \quad V = 29.5 \text{ kips} \]
Shear Rupture Capacity > Applied Force, UCV = 0.461, OK

b. Shear Rupture Capacity due to Shear Load (Primary Side)
Net Shear Area,
\[ \text{Anv} = (L - nr \cdot hdv) \cdot t \]
Number of Areas in Consideration,
\[ n_1 = n \]
Shear Rupture Capacity, (J4-4)
\[ \text{Rvr}_2 = \Lambda \cdot v_r \cdot n_1 \cdot 0.6 \cdot F_u \cdot \text{Anv} \]
\[ \text{Rvr}_2 = 63.993 \text{ kips} \quad V = 29.5 \text{ kips} \]
Shear Rupture Capacity > Applied Force, UCV = 0.461, OK
Governing Shear Rupture Capacity,
\[ \text{Rvr} = \min(\text{Rvr}_1, \text{Rvr}_2) \]
\[ \text{Rvr} = 63.993 \text{ kips} \quad V = 29.5 \text{ kips} \]
Shear Rupture Capacity > Applied Force, UCV = 0.461, OK

4. Block Shear Capacity
(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)
a. Block Shear Capacity due to Shear Load (Secondary Side)
Reduction Factor,
\[ n_v = 1 \]
\[ U_{bs} = 1.0 \quad (\text{tension stress is uniform}) \]
Gross Shear Area,
\[ \text{Agv} = [(nr - 1 \cdot s \cdot \text{Lev}) \cdot t] \]
\[ \text{Agv} = 2.269 \text{ in}^2 \]
Net Tension Area,
\[ \text{Ant} = [\text{Leh} + (n_v - 1) \cdot sv - (n_v - 0.5) \cdot \text{hdh}] \cdot t \]
\[ \text{Ant} = 0.254 \text{ in}^2 \]
Net Shear Area,
\[ \text{Anv} = \text{Agv} - [(nr - 0.5) \cdot \text{hdv}] \cdot t \]
\[ \text{Anv} = 1.585 \text{ in}^2 \]
Number of Areas in Consideration,
nl = n

Block Shear Capacity, (J4-5)

\[
R_{bs1} = \Lambda_{bs} \cdot n \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant, 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)
\]

\[R_{bs1} = 63.766 \text{ kips}\]

**Block Shear Capacity > Applied Force, UCV = 0.463, OK**

b. Block Shear Capacity due to Shear Load (Primary Side)

Reduction Factor,

\[Ubs = 1.0\]

(tension stress is uniform)

Gross Shear Area,

\[Agv = [(nr - 1) \cdot s + Lev] \cdot t\]

\[Agv = 2.269 \text{ in}^2\]

Net Tension Area,

\[Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t\]

\[Ant = 0.303 \text{ in}^2\]

Net Shear Area,

\[Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t\]

\[Anv = 1.585 \text{ in}^2\]

Number of Areas in Consideration,

\[nl = n\]

Block Shear Capacity, (J4-5)

\[
R_{bs2} = \Lambda_{bs} \cdot nl \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant, 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)
\]

\[R_{bs2} = 66.602 \text{ kips}\]

**Block Shear Capacity > Applied Force, UCV = 0.443, OK**

Governing Block Shear Capacity,

\[R_{bs} = \min(R_{bs1}, R_{bs2})\]

\[R_{bs} = 63.766 \text{ kips}\]

**Block Shear Capacity > Applied Force, UCV = 0.463, OK**

D. CONNECTION ANGLE TO GIRDER WEB CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.6, pages 16.1-125)

Shear Capacity Per Bolt,

\[\Lambda_{rv} = 11.928 \text{ kips}\]

Bolt Shear Capacity,

\[R_b = n \cdot nb \cdot \Lambda_{rv}\]

\[R_b = 71.569 \text{ kips}\]

**Bolt Shear Capacity > Applied Force, UCV = 0.412, OK**

2. Check for Spacing
Connection Angle Thickness,
\[ t_1 = 0.313 \text{ in} \]

Girder Web Thickness,
\[ t_2 = 0.295 \text{ in} \]

Vertical Spacing of Bolts,
\[ s = 3 \text{ in} \]
\[ s_{\text{min}} = 2 \cdot \frac{2}{3} \cdot db \quad s_{\text{min}} = 2 \text{ in} \]
\[ s_{\text{max}} = \text{min}(12\text{ in}, 24 \cdot \text{min}(t_1, t_2)) \quad s_{\text{max}} = 7.08 \text{ in} \]

Spacing > Min. Spacing & Spacing < Max. Spacing, OK

3. Check for Edge Distance

(AISC 14th Ed. Specifications, Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Connection Angle Thickness,
\[ t_1 = 0.313 \text{ in} \]

Connection Angle Edge Distances,
\[ L_{v1} = 1.25 \text{ in} \]
\[ L_{h1} = 1.5 \text{ in} \]

Vertical Edge Distance,
\[ L_{\text{vcon}} = \frac{L_{v1}}{L_{v2}} \quad L_{\text{vcon}} = \left( \frac{1.25 \text{ in}}{\text{NA}} \right) \]
\[ L_{\text{vmin}} = \frac{L_{\text{vmin1}}}{L_{\text{vmin2}}} \quad L_{\text{vmin}} = \left( \frac{1 \text{ in}}{\text{NA}} \right) \]
\[ \text{min}(L_{\text{vcon}}) = L_{v1} \]
\[ L_{\text{vmax}} = \text{min}(6\text{ in}, 12 \cdot t_1) \]
\[ L_{\text{vmax}} = 3.756 \text{ in} \]

Edge Distance ≥ Min. Edge Distance & Edge Distance ≤ Max. Edge Distance, OK

Horizontal Edge Distance,
\[ L_{\text{hcon}} = \frac{L_{h1}}{L_{h2}} \quad L_{\text{hcon}} = \left( \frac{1.5 \text{ in}}{\text{NA}} \right) \]
\[ L_{\text{hmin}} = \frac{L_{\text{hmin1}}}{L_{\text{hmin2}}} \quad L_{\text{hmin}} = \left( \frac{1.125 \text{ in}}{\text{NA}} \right) \]
\[ \text{min}(L_{\text{hcon}}) = L_{h1} \]
\[ L_{\text{hmax}} = \text{min}(6\text{ in}, 12 \cdot t_1) \]
\[ L_{\text{hmax}} = 3.756 \text{ in} \]
E. GIRDER WEB CHECK

1. Bolt Capacity


Total Force Acting Per Bolt,

\[ V_1 = \frac{V}{n_r \cdot n} \]

\[ V_1 = 4.917 \text{ kips} \]

\[ nr_2 = 0 \]

\[ V_2 = 0 \text{ kips} \]

\[ V_{12} = V_1 + V_2 \]

\[ V_{12} = 4.917 \text{ kips} \]

Effective Thickness of Web,

\[ \text{tweff} = \frac{V_1}{V_{12}} \]

\[ \text{tweff} = 0.295 \text{ in} \]

Bearing Area,

\[ A_{brg} = d_b \cdot \text{tweff} \]

\[ A_{brg} = 0.221 \text{ in}^2 \]

Allowable Bearing Strength Using Edge Distance, \((J3-6a, J3-6c)\)

\[ h_{hdh} < h_{dl} \]

\[ F_{be} = A_{brg} \cdot F_u \cdot 2.4 \cdot A_{brg} \]

\[ F_{be} = 17.258 \text{ kips} \]

Allowable Bearing Strength Using Bolt Spacing, \((J3-6a, J3-6c)\)

\[ h_{hdh} < h_{dl} \]

\[ F_{bs} = A_{brg} \cdot F_u \cdot \min(1.2 \cdot (s - h_{dv}) \cdot \text{tweff}, 2.4 \cdot A_{brg}) \]

\[ F_{bs} = 17.258 \text{ kips} \]

Number of Areas in Consideration,

\[ n_1 = n \]

Connection Angle,

\[ n_2 = n \]

Bolt Capacity,

\[ R_{brg} = n_v \cdot \left( \min(n_1 \cdot F_{be}, n_2 \cdot A_{rv}) + \min(n_1 \cdot F_{bs}, n_2 \cdot A_{rv}) \cdot (n_r - 1) \right) \]

\[ R_{brg} = 71.569 \text{ kips} \]

\[ V = 29.5 \text{ kips} \]

Bolt Capacity > Applied Force, \(UCV = 0.412, \text{ OK}\)
III. DETAILS

A. SKETCH

Note: Figure above does not present actual design. Refer to connection schedule.

SHEAR CONNECTION: W BEAM WITH DOUBLE ANGLE (BOLTED/BOLTED)
ONE-WAY SHEAR CONNECTION TO W GIRDER WEB
B. CONNECTION DETAILS

<table>
<thead>
<tr>
<th>Mark</th>
<th>Size</th>
<th>Grade</th>
<th>girder Mark Size Grade g</th>
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</thead>
<tbody>
<tr>
<td>B10159</td>
<td>W12X40</td>
<td>A992</td>
<td>5 1/4&quot;</td>
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<table>
<thead>
<tr>
<th>db</th>
<th>Bolt Type</th>
<th>Remarks</th>
<th>nr</th>
<th>s</th>
<th>nv</th>
<th>sv</th>
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<tbody>
<tr>
<td>3/4&quot;</td>
<td>A325-N</td>
<td>Short Slot on Outstanding Leg of Angle Only</td>
<td>3</td>
<td>3&quot;</td>
<td>1</td>
<td>0&quot;</td>
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<table>
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<th>o2</th>
<th>D</th>
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<th>dcT</th>
<th>cT</th>
<th>dcB</th>
<th>cB</th>
<th>Cut Flush Case</th>
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<tbody>
<tr>
<td>3&quot;</td>
<td>4&quot;</td>
<td>2 3/4&quot;</td>
<td>4&quot;</td>
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# Bolts at Beam Web

<table>
<thead>
<tr>
<th>db</th>
<th>Bolt Type</th>
<th>Remarks</th>
<th>nr</th>
<th>s</th>
<th>nv</th>
<th>sv</th>
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</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>A325-N</td>
<td>Short Slot on Outstanding Leg of Angle Only</td>
<td>3</td>
<td>3&quot;</td>
<td>1</td>
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## Connection Angle

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<tr>
<th>Size</th>
<th>Grade</th>
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<th>leg1</th>
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## Horizontal Stiffener Plate (As Req’d)

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<th>t</th>
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<th>L</th>
<th>w13</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
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<td>A572-50</td>
<td>2 5/8&quot;</td>
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<td>Horizontal Stiffener Plate is Required</td>
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## Governing Limit State of SC Connection

<table>
<thead>
<tr>
<th>V</th>
<th>Connection Capacity</th>
<th>UCV</th>
<th>Governing Check</th>
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<tbody>
<tr>
<td>29.5 kips</td>
<td>40.725 kips</td>
<td>0.724</td>
<td>Flexural Cope</td>
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<tr>
<td></td>
<td></td>
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<td>Buckling Capacity of Beam</td>
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### Remarks on Connection / Connecting Elements

<table>
<thead>
<tr>
<th>For Bolts</th>
<th>For Connector Thickness</th>
<th>For Connector Length</th>
<th>For Bolt Spacing</th>
<th>For Edge Distance</th>
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<td>OK</td>
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</table>

### Remarks on Beam Web / Girder Web

<table>
<thead>
<tr>
<th>For Beam Web</th>
<th>For Girder Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK</td>
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IV. REFERENCES