SHEAR CONNECTION: W BEAM WITH SHEAR PLATE ONE-WAY SHEAR CONNECTION TO W COLUMN WEB

Note: Figure above does not represent actual design. Refer to connection schedule.
# I. DESIGN DATA AND LOADS (ASD-14th Edition)

## COLUMN PROPERTIES: W14X90 - A992

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth, d</td>
<td>14 in</td>
</tr>
<tr>
<td>Flange Width, bf</td>
<td>14.5 in</td>
</tr>
<tr>
<td>Distance k, k</td>
<td>2 in</td>
</tr>
<tr>
<td>Area, Ag</td>
<td>26.5 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>
</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>
</tbody>
</table>

## Top of Steel Elevation, Elev = 0 ft + 0 in

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Length, L</td>
<td>30 ft</td>
</tr>
<tr>
<td>Slope, θsl</td>
<td>0 deg</td>
</tr>
<tr>
<td>Depth of Top Cope, dcT</td>
<td>0 in</td>
</tr>
<tr>
<td>Length of Top Cope, cT</td>
<td>0 in</td>
</tr>
</tbody>
</table>

## BOLTS PROPERTIES:

### 3/4" ø - A325-N

- **Bolt Diameter, db**: 0.75 in
- **Bolt Shear Strength, Arv**: 11.928 kips
- **Bolt Type, Bolt_Type**: A325-N
- **Bolt Tensile Strength, Arn**: 19.88 kips
- **Connection Type, Conn_type**: Bearing Type

### For Shear Plate to Beam Web Connection:

- **Bolt Diameter, db**: 0.75 in
- **Bolt Shear Strength, Arv**: 11.928 kips
- **Bolt Tensile Strength, Arn**: 19.88 kips
- **Bolt Type, Bolt_Type**: A325-N
- **Connection Type, Conn_type**: Bearing Type

### BOLTS PROPERTIES:
Number of Bolt Rows, \( nr = 4 \)

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

Total Number of Bolts \( (nr \cdot nv) \), \( nb = 4 \)

Holes at Beam Web, Vertical Hole Dimension, \( h_{dv} = 0.875 \) in

Holes at Shear Plate, Vertical Hole Dimension, \( h_{dv} = 0.875 \) in

Bolt First Down from Top of Beam, Vertical Edge Distance \( (D-dcT) \), \( Lev = 3 \) in

Horizontal Edge Distance, \( Leh = 1.75 \) in

For Shear Plate to Column Web Connection:

Preferred Weld Size \( (w_1) \), \( w = 0.25 \) in

SAFETY AND RESISTANCE FACTORS:

Safety Factor, \( \Omega \text{(ASD)} \)

Resistance Factor, \( \phi \text{(LRFD)} \)

Modification Factor, \( \Lambda \)

\[
\Lambda = \frac{1}{\Omega} \quad \text{(if ASD)}
\]

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

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 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 \( C, WT, L\text{(v)} \),

\[
\Omega_{v} = 1.67 \quad \phi_{v} = 0.90 \quad \Lambda_{v} = 0.60
\]
For Shear Rupture (vr),
\[ \Omega_{vr} = 2.00 \quad \Phi_{vr} = 0.75 \]  \[ \Lambda_{vr} = 0.50 \]

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

APPLIED LOADS:

Given End Reaction

Beam:
Shear Load, \[ V = 10 \text{ kips} \]
Adjacent Shear Load (if any), \[ V_2 = 0 \text{ kips} \]
II. CALCULATIONS

A. BEAM WEB CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-
Bolt Capacity due to Shear Load

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

Bolt Centerline Distance from Face of Support,
\[ ab = 0.5 \cdot (bf - tw) + \text{gap} + \text{Leh} + 0.5(nv - 1) \cdot sv \frac{\cos(\theta_{sk})}{\cos(\theta_{sk})} \]
\[ ab = 9.28 \text{ in} \]

Eccentricity Distance of End Reaction from Bolt Line,
\[ ebv = ab \]
\[ ebv = 9.28 \text{ in} \]

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

Eccentric Load Coefficient,
(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-
\[ C = 1.172 \]

Allowable Bearing Strength Using Edge Distance, (J3-6a, J3-6c)
\[ \text{hDH} < \text{hDLS} \]
\[ F_{be} = \text{Abrg} \cdot F_{u} \cdot \left[ 1.2 \cdot (\text{Lev} - 0.5 \cdot \text{hDV}) \cdot \text{tw} \right] \]
\[ 1.2 \cdot (\text{Leh} - 0.5 \cdot \text{hDH}) \cdot \text{tw} \]
\[ 2.4 \cdot \text{Abrg} \]
\[ ebv > 0 \text{ in} \]
\[ F_{be} = \min (F_{be}) \]
\[ F_{be} = 12.797 \text{ kips} \]

Allowable Bearing Strength Using Bolt Spacing (J3-6a, J3-6c)
\[ \text{hDh} < \text{hDLS} \]
\[ F_{bs} = \text{Abrg} \cdot F_{u} \cdot \min [1.2 \cdot (s - \text{hDV}) \cdot \text{tw} , 2.4 \cdot \text{Abrg}] \]
\[ F_{bs} = 14.625 \text{ kips} \]

Number of Area in Consideration,
\[ n_{1} = 1 \]

Bolt Bearing Capacity,
\[ ebv > 0 \text{ in} \]
\[ R_{brg} = C \cdot \min (n_{1} \cdot F_{be}, n_{1} \cdot F_{bs}, n \cdot A_{rv}) \]
Rbrg = 13.977 V = 10 kips

Bolt Capacity > Applied Force, UCV = 0.715, OK

2. Shear Capacity


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

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

Clear Distance Between Transverse Stiffeners,
\[ a = 0 \text{ in} \]

Web Plate Buckling Coefficient, (G2-6)
\[ htw < 260 \]
\[ kv = 5 \]

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)
\[ Rv = A_{bh} \cdot 0.6 \cdot Fy \cdot d \cdot tw \cdot Cv \]
\[ Rv = 70.509 \text{ kips} \]
\[ V = 10 \text{ kips} \]

Shear Capacity of Section > Applied Force, UCV = 0.142, OK

B. BEAM WEB TO SHEAR PLATE 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,
\[ Rb = n \cdot C \cdot Arv \]
\[ Rb = 13.977 \text{ kips} \]
\[ V = 10 \text{ kips} \]

Bolt Shear Capacity > Applied Force, UCV = 0.715, OK

2. Check for Spacing
Shear Plate Thickness,
\[ t_1 = 0.375 \text{ in} \]

Beam Web Thickness,
\[ t_2 = 0.25 \text{ in} \]

Vertical Spacing of Bolts,
\[ s = 3 \text{ in} \]
\[ s_{\text{min}} = 2 \left( \frac{2}{3} \right) \cdot db \]
\[ s_{\text{max}} = \min(12\text{in}, 24 \cdot \min(t_1, t_2)) \]
\[ s_{\text{max}} = 6 \text{ in} \]

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

3. Check for Edge Distance

Shear Plate Thickness,
\[ t_1 = 0.375 \text{ in} \]

Shear Plate Edge Distances,
\[ L_{\text{lev1}} = 1.25 \text{ in} \]
\[ L_{\text{leh1}} = 1.25 \text{ in} \]

Beam Web Thickness,
\[ t_2 = 0.25 \text{ in} \]

Beam Web Edge Distances,
\[ L_{\text{lev2}} = \text{NA} \]
\[ L_{\text{leh2}} = 1.75 \text{ in} \]

Vertical Edge Distance,
\[ L_{\text{LEVCON}} = \left( \frac{L_{\text{LEV1}}}{L_{\text{LEV2}}} \right) \]
\[ L_{\text{LEVMIN}} = \left( \frac{L_{\text{LEVMIN1}}}{L_{\text{LEVMIN2}}} \right) \]
\[ \min(L_{\text{LEVCON}}) = L_{\text{LEV1}} \]
\[ L_{\text{LEVMAX}} = \min(6\text{in}, 12 \cdot t_1) \]
\[ L_{\text{LEVMAX}} = 4.5 \text{ in} \]

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

Horizontal Edge Distance,
Lehcon = \begin{pmatrix} Leh1 \\ Leh2 \end{pmatrix} \quad Lehcon = \begin{pmatrix} 1.25 \text{ in} \\ 1.75 \text{ in} \end{pmatrix}

Lehmin = \begin{pmatrix} Lehmin1 \\ Lehmin2 \end{pmatrix} \quad Lehmin = \begin{pmatrix} 1.125 \text{ in} \\ 1 \text{ in} \end{pmatrix}

\min(Lehcon) = Leh1

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

Lehmax = 4.5 \text{ in}

\text{Edge Distance} \geq \min. \text{ Edge Distance} \& \text{ Edge Distance} \leq \max. \text{ Edge Distance}, \text{ OK}

C. SHEAR PLATE CHECK

1. Check for Maximum Thickness

(AISC 14th Ed. Manual, Part 10, page 10-104)

Exceptions for \(nv = 1\) and \(nv = 2\),

Shear Plate,

\[ t \leq \frac{db}{2} + \frac{1}{16} \]

\[ Leh \geq 2 \cdot db \]

Beam Web,

\[ tw \leq \frac{db}{2} + \frac{1}{16} \]

\[ Leh \geq 2 \cdot db \]

**Check maximum thickness of plate**

Coefficient for Eccentrically Loaded Bolts,


\[ C' = 11.256 \text{ in} \]

Area of Bolts,

\[ Ab = \frac{n \cdot db^2}{4} \quad Ab = 0.442 \text{ in}^2 \]

Length of Plate,

\[ L = (nr-1) \cdot s + 2 \cdot Lev \quad L = 11.5 \text{ in} \]

Maximum Thickness,

\[ t_{max} = \frac{6 \cdot \left( Env \cdot Ab \cdot C' \right)}{0.9 \cdot \left( Ab \cdot C' \right)} \]

\[ Fy \cdot L^2 \]

\[ t_{max} = 0.376 \text{ in} \quad t = 0.375 \text{ in} \]

**Plate thickness < Maximum Thickness Permitted, OK**

Governing Shear Plate Thickness,

Case = Maximum thickness need be checked
t \leq t_{max}
tg = t

tg = 0.375 \text{ in}
2. Check for Stiffener Plate Requirement


Distance of First Bolt Line from the Face of Support,
\[ ab_1 = \frac{0.5(b_f - t_w) + \text{gap}}{\cos(\theta_{sk})} + \text{Leh} \quad ab_1 = 9.28 \text{ in} \]

Lateral Displacement Capacity, (10-6)
\[ R_{req} = ab_1 \cdot 1500 \cdot \frac{L \cdot \tan \theta_{ab}}{ab_1^2} \cdot 1 \text{ ksi} \]
\[ R_{req} = 19.87 \text{ in} \quad V = 10 \text{ kips} \]

Lateral Displacement Capacity > Applied Force, \(UCV = 0.503, OK\)

Check for Requirement of Stiffener Plates,
\[ \eta = \frac{R_{req}}{V} \quad \eta = 1.987 \]

Stiffener Plate is not required. OK

3. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-

Bolt Capacity due to Shear Load

Bearing Area,
\[ Ab_{rg} = db \cdot \tan \theta_{rg} \quad Ab_{rg} = 0.281 \text{ in}^2 \]

Bolt Centerline Distance from Face of Support,
\[ ab = \frac{0.5(b_f - t_w) + \text{gap} + \text{Leh} + 0.5(n_v - 1) \cdot sv}{\cos(\theta_{sk})} \quad ab = 9.28 \text{ in} \]

Eccentricity Distance of End Reaction from Bolt Line,
\[ eb_v = ab \quad eb_v = 9.28 \text{ in} \]

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

Eccentric Load Coefficient,
(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-
\[ C = 1.172 \]
Allowable Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

\[ F_{be} = A_{brg} \cdot F_y \cdot \left[ 1.2 \cdot (L_e - 0.5 \cdot h_{dv}) \cdot t_g \right] \]

\[ F_{be} = \min (F_{be}) \]

\[ F_{be} = 9.38 \text{ kips} \]

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

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

\[ F_{bs} = 19.575 \text{ kips} \]

Number of Area in Consideration,
\[ n_l = n \]

Bolt Bearing Capacity,
\[ R_{brg} = C \cdot \min (n_l \cdot F_{be}, n_l \cdot F_{bs}, n \cdot A_{rv}) \]

\[ R_{brg} = 10.99 \text{ kips} \]

Bolt Capacity > Applied Force, UCV = 0.91, OK

4. Shear Yielding Capacity

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

a. Shear Yielding Capacity due to Shear Load

Length,
\[ L = (n_r - 1) \cdot s + 2 \cdot L_e \]

\[ L = 11.5 \text{ in} \]

Erection Stability,


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

Number of Areas in Consideration,
\[ n_l = n \]

Shear Yielding Capacity, (J4-3)

\[ R_{vy} = A_{vy} \cdot n_l \cdot 0.6 \cdot F_y \cdot L \cdot t_g \]

\[ R_{vy} = 62.1 \text{ kips} \]

Shear Yielding Capacity > Applied Force, UCV = 0.161, OK

5. Rupture Capacity
a. Shear Rupture Capacity due to Shear Load

Net Shear Area,

\[ \text{Anv} = (L - nr \cdot hdv) \cdot \text{tg} \]

Number of Areas in Consideration,

\[ n_1 = n \]

Shear Rupture Capacity, (J4-4)

\[ \text{Rvr} = \text{Avr} \cdot n_1 \cdot 0.6 \cdot \text{Fu} \cdot \text{Anv} \]

\[ \text{Rvr} = 52.2 \text{ kips} \]

\[ \text{V} = 10 \text{ kips} \]

Shear Rupture Capacity > Applied Force, UCV = 0.192, OK

6. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

\[ n_v = 1 \]

\[ \text{Ubs} = 1.0 \]

(tension stress is uniform)

Gross Shear Area,

\[ \text{Agv} = [(nr - 1 \cdot s + Lev) \cdot \text{tg} \]

\[ \text{Agv} = 3.844 \text{ in}^2 \]

Net Tension Area,

\[ \text{Ant} = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot \text{tg} \]

\[ \text{Ant} = 0.27 \text{ in}^2 \]

Net Shear Area,

\[ \text{Anv} = \text{Agv} - [(nr - 0.5) \cdot hdv] \cdot \text{tg} \]

\[ \text{Anv} = 2.695 \text{ in}^2 \]

Number of Areas in Consideration,

\[ n_1 = n \]

Block Shear Capacity, (J4-5)

\[ \text{Rbs} = \text{Abs} \cdot n_1 \cdot \text{min}(0.6 \cdot \text{Fu} \cdot \text{Anv} + \text{Ubs} \cdot \text{Fu} \cdot \text{Ant}, 0.6 \cdot \text{Fy} \cdot \text{Agv} + \text{Ubs} \cdot \text{Fu} \cdot \text{Ant}) \]

\[ \text{Rbs} = 49.329 \text{ kips} \]

\[ \text{V} = 10 \text{ kips} \]

Block Shear Capacity > Applied Force, UCV = 0.203, OK

7. Local Buckling Capacity


Distance of Bolt Line to Support,

Stiffeners = Required

\[ ab = \text{gap} + \text{Leh} \]

\[ ab = 9.5 \text{ in} \]
Coefficient, \[ \lambda = \frac{L \cdot F_y}{10 \cdot \tan \left( 475 + 280 \left( \frac{L}{ab} \right)^{0.5} \right) \text{kSI}^{0.5}} \]

\[ \lambda = 0.618 \]

\[ \lambda > 1.41 \]

\[ Q = \frac{1.30}{\lambda^2} \]

Allowable Flexural Local Buckling Stress or Yielding Stress, \[ F_{cr} = Q \cdot F_y \]

Gross Plastic Section Modulus,

\[ Z_x = \left( \frac{\tan L^2}{4} \right) \]

Eccentricity,

\[ e = ab \]

Local Buckling Capacity,

\[ R_{bc} = \frac{A_{bc} \cdot F_{cr} \cdot 2x}{e} \]

\[ R_{bc} = 28.134 \text{ kips} \]

Local Buckling Capacity will not control, OK

8. Flexural Rupture Capacity


(AISC 14th Ed. Steel Construction Manual Design Examples, page IIA-104)

Net Plastic Section Modulus,

\[ \text{mod}(nr,2) \leq 0 \]

\[ Z_{net} = \left( \frac{\tan L^2}{4} - \frac{\tan \cdot \text{hdv} \cdot nr^2 \cdot s}{4} \right) \]

\[ Z_{net} = 8.461 \text{ in}^3 \]

Flexural Rupture Capacity,

\[ R_{fr} = \frac{A_{fr} \cdot F_u \cdot Z_{net}}{e} \]

\[ R_{fr} = 25.828 \text{ kips} \]

Flexural Rupture Capacity > Applied Force, UCV = 0.387, OK

9. Flexural Yielding Capacity with Von-Mises Shear Reduction

Flexural Yielding Capacity,

\[ R_{fc} = \frac{A_b \cdot F_y \cdot L \cdot t_g}{\sqrt{2.25 + 16 \cdot \left( \frac{e}{L} \right)^2}} \]

\[ R_{fc} = 25.618 \text{ kips} \quad V = 10 \text{ kips} \]

Flexural Yielding Capacity > Applied Force, UCV = 0.39, OK

10. Interaction of Shear Yielding, Shear Buckling, and Flexural Yielding Capacities

(AISC 14th Ed. Manual Part 10, pages 10-104 to 10-105)

From AISC Manual Equation 10-5,

\[ \left( \frac{V_r}{V_c} \right)^2 + \left( \frac{M_r}{M_c} \right)^2 \leq 1.0 \]

\[ V_r = V \quad V_r = 10 \text{ kips} \]

\[ M_r = V_r \cdot e \quad M_r = 95 \text{ kips} \cdot \text{in} \]

Shear Yielding Capacity,

\[ V_c = A_{vy} \cdot 0.6 \cdot F_y \cdot L \cdot t_g \quad V_c = 62.1 \text{ kips} \]

Flexural Yielding Capacity,

\[ M_c = A_b \cdot F_y \cdot 2x \quad M_c = 267.272 \text{ kips} \cdot \text{in} \]

Interaction, (10-5)

\[ UCV = \left( \frac{V_r}{V_c} \right)^2 + \left( \frac{M_r}{M_c} \right)^2 \]

\[ UCV = 0.152 \]

Interaction < 1.0, UCV = 0.152, OK

D. SHEAR PLATE TO COLUMN WEB CHECK

1. Welds Check


Number of Weld Sides,

\[ nws = 2 \]

Minimum Weld Size,

\[ w_{min} = 0.25 \text{ in} \quad w = 0.25 \text{ in} \]

Preferred Weld Size = Minimum Weld Size, OK
III. DETAILS
A. SKETCH

SHEAR CONNECTION: W BEAM WITH SHEAR PLATE ONE-WAY SHEAR CONNECTION TO W COLUMN WEB

Note: Figure above does not represent actual design. Refer to connection schedule.
## B. CONNECTION SCHEDULE

### Column

<table>
<thead>
<tr>
<th>Mark</th>
<th>Size</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1000</td>
<td>W14X90</td>
<td>A992</td>
</tr>
</tbody>
</table>

### Beam

<table>
<thead>
<tr>
<th>Mark</th>
<th>Size</th>
<th>Grade</th>
<th>gap</th>
<th>θsl</th>
<th>θsk</th>
<th>D</th>
<th>Leh</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1000</td>
<td>W16X26</td>
<td>A992</td>
<td>1/2”</td>
<td>0°</td>
<td>0°</td>
<td>3”</td>
<td>1 3/4”</td>
</tr>
</tbody>
</table>

### Cope Dimension

<table>
<thead>
<tr>
<th>dcT</th>
<th>cT</th>
<th>dcB</th>
<th>cB</th>
<th>Cut</th>
<th>Flush</th>
<th>Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>0”</td>
<td>0”</td>
<td>0”</td>
<td>0”</td>
<td>NR</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 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>
</tr>
</thead>
<tbody>
<tr>
<td>3/4”</td>
<td>A325-N</td>
<td>Short-Slot on Shear Plate ONLY</td>
<td>4</td>
<td>3”</td>
<td>1</td>
<td>0”</td>
</tr>
</tbody>
</table>
### Shear Plate

<table>
<thead>
<tr>
<th>t</th>
<th>Grade</th>
<th>Lev</th>
<th>Leh</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot;</td>
<td>A36</td>
<td>1 1/4&quot;</td>
<td>1 1/4&quot;</td>
</tr>
</tbody>
</table>

### Remarks as per AISC Equation 10-5

Interaction < 1.0, UCV = 0.152, OK

### Governing Limit State of SC Connection

<table>
<thead>
<tr>
<th>V</th>
<th>Connection Capacity</th>
<th>UCV</th>
<th>Governing Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 kips</td>
<td>10.990 kips</td>
<td>0.910</td>
<td>Bolt Capacity at Shear Plate</td>
</tr>
</tbody>
</table>

### 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>
</tr>
</thead>
<tbody>
<tr>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
</tbody>
</table>
### Remarks on Connection / Connecting Elements

<table>
<thead>
<tr>
<th>For Weld on Shear Plate to Column Web</th>
<th>For Weld on Shear Plate to Stiffener</th>
<th>For Weld on Stiffener Plate to Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>OK</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

### Remarks on Beam Web / Column Web

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