1 Overview

The purpose of Standard Bridge Drawing, GSD-1-19, is to address updated design requirements for intermediate crossframes not previously addressed in the GSD-1-96 Standard Bridge Drawing. An intermediate diaphragm was added for shallow, rolled beam members. End crossframes and scupper details remain unchanged from GSD-1-96.

This drawing includes four separate plan sheets, which separately address the following steel bridge details:

Sheet 1/4: Intermediate Crossframes – This drawing addresses standard intermediate crossframes, for use with tangent steel beams or girders designed with “line-girder” methodologies per the ODOT Bridge Design Manual.

Sheet 2/4: Intermediate Diaphragms - This drawing addresses standard intermediate diaphragms, for use with tangent steel rolled beams (≤ 36” deep) designed with “line-girder” methodologies per the ODOT Bridge Design Manual.

Sheet 3/4: End Crossframe Details – This drawing remains unchanged from Standard Bridge Drawing GSD-1-96.

Sheet 4/4: Scupper Details - This drawing remains unchanged from Standard Bridge Drawing GSD-1-96.

2 Plan Preparation Requirements:

2.1 Sheet 1/4 – Intermediate Crossframes

2.1.1 Limitations for Intermediate Crossframes

This drawing provides standard intermediate crossframe details for use with tangent steel beams or girders designed with “line-girder” methodologies per the ODOT Bridge Design Manual.

The intermediate crossframes are applicable to rolled beam depths or girder web depths between 36” and 72” as well as the spacing and overhang limits shown on the sheet.

Crossframe Spacing: The intermediate crossframe designs are applicable for intermediate crossframe spacings up to 25 feet for positive moment regions and up to 15 feet for negative moment regions for bridge widths up to 70 feet wide. For bridge decks wider than 70 feet, the intermediate crossframe designs are applicable for spacings up to 21 feet for positive moment regions and up to 15 feet for negative moment regions.
2.1.2 Intermediate Crossframe Types

There are three separate types of intermediate crossframes, considered equivalent for design purposes and each with different fabrication and constructability pros and cons. Unless the contract plans specify otherwise, the contractor may choose either the Type A, B or C intermediate crossframes in accordance with the limitations specified in GSD-1-19. The three types of intermediate crossframes include:

Type A: The National Steel Bridge Alliance (NSBA) preferred crossframe, includes crossframe angles welded to gusset plates, which are field bolted to beam/girder stiffener connection plates. This is NSBA’s preferred type of crossframe for fabrication and erection efficiencies. Depending on beam/girder depth and deck overhang width, crossframe angle size and number of bolts required vary.

Type B: The field-welded intermediate crossframe, similar to the Type 3 intermediate crossframe from GSD-1-96, includes an erection bolt for steel beam/girder erection and field welded connection prior to the deck pour. Limitations include angle size and minimum weld length, depending on beam/girder depth and deck overhang width.

Type C: The field-bolted, knock-down type crossframe, similar to the Type 4 intermediate crossframe from GSD-1-96, is fully bolted prior to the deck pour. Use of this type of intermediate crossframe is not permitted for shallow beams/girders with larger overhangs and/or wider beam/girder spacings. The number of bolts required varies depending on deck overhang width and beam/girder depth.

2.2 Sheet 2/4 - Intermediate Diaphragms

Sheet 2/4: Intermediate Diaphragms - This drawing addresses standard intermediate diaphragms, for use with tangent steel rolled beams up to 36” deep that are designed with “line-girder” methodologies per the ODOT Bridge Design Manual.

Beam Spacing: The standard intermediate diaphragms are applicable to beam spacings up to 10.5 feet.

Diaphragm Spacing: The intermediate diaphragm designs are applicable for diaphragm spacings up to 25 feet in positive moment regions and up to 15 feet in negative moment regions for bridge decks up to 70 feet wide. For bridge decks wider than 70 feet, the intermediate diaphragm designs are applicable for spacings up to 21 feet for positive moment regions and up to 15 feet for negative moment regions due to the influence from the increasing ODOT finishing machine loading.
2.3 Estimated Quantities

Assume the use of the Type A Crossframes to determine the total weight of structural steel for the Estimated Quantities. If the pay unit is Lump Sum, provide the estimated weight on the Estimated Quantities Form as defined in Section 1504 of the L&D Manual, Vol. 3. If the pay unit is Pound, provide the following note on the Estimated Quantities plan sheet:

Item 513 Structural Steel Members, Level __:

This Total Weight is based on the use of Type A Crossframes. Provide the unit cost for Structural Steel using the total weight provided, regardless of any change to the total weight resulting from the selection of Type B or Type C crossframes in lieu of Type A.

3 Detail Information

3.1 Design Criteria

The intermediate crossframes and intermediate diaphragms were designed in accordance with the following references:

- AASHTO Guide Design Specifications for Bridge Temporary Works, 2nd Ed. [GSBTW]
- Ohio Bridge Design Manual, 2007, including the 2018 updates [ODOT BDM]
- ASCE 7-10 Minimum Design Loads and Associated Criteria for Buildings and Other Structures [ASCE]

3.2 Design Methodology

Intermediate crossframes and diaphragms were designed for the following:

- Externally applied forces
  - Wind loads – ASCE (Construction), AASHTO (Final)
  - Cantilever bracket loading during deck construction – GSBTW
- Bracing forces
  - Based on in-plane bending moment of the beam/girder. – FHWA
  - Can be simplified to 0.02 $M_{br}$ ($M_{br}$ = in-plane bending moment of the beam/girder) based on AISC Steel Construction Manual, 15th Edition, App. 6.3.2a.
- Bracing Stiffness – FHWA

The cantilever bracket used for construction of the deck overhang creates a torsional force in the beam/girder that is carried by the intermediate bracing. This cantilever bracket loading governs the bracing design for large deck overhangs. Loads for the cantilever bracket during the deck pour were developed as follows:
• Dead loads plus allowance for form loads – GSBTW
• Horizontal Construction Load – GSBTW
• Area live loads – GSBTW
• Deck finishing machine loads per ODOT BDM

3.3 Sample Design Calculations

Refer to the Appendix for a sample bracing calculation. The sample calculation provided is for a Type B Intermediate Crossframe. The Type B Crossframe is temporarily connected with an erection bolt, then field-welded prior to deck construction. The design of Type A and Type C crossframes are similar, but with the design of the bolted connections.
Appendix

Sample Calculations for Type B Intermediate Crossframe
### REFERENCES

<table>
<thead>
<tr>
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<th>Abbreviation</th>
<th>Title</th>
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<tr>
<td>2</td>
<td>ODOT BDM</td>
<td>ODOT Bridge Design Manual, 2007</td>
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<td>ASCE 7</td>
<td>ASCE 7-10 Minimum Design Loads and Associated Criteria for Buildings and Other Structures</td>
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<td>ODOT SBR-2-13</td>
<td>ODOT Standard Bridge Drawing 57&quot; Single Slope Concrete Median Bridge Railing</td>
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<td>7</td>
<td>Battistini</td>
<td>Stiffness Behavior of Cross Frames in Steel Bridge Systems</td>
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<td>9</td>
<td>CalTrans</td>
<td>CalTrans Bridge Design Aids Table of Maximum Moments, Shears, and Reactions Simple Spans</td>
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<td>AISC Mom.</td>
<td>AISC Moments Shears and Reactions for Continuous Highway Bridges</td>
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<td>12</td>
<td>NSBA SS</td>
<td>NSBA Short Span Standards, Jan 2013</td>
</tr>
<tr>
<td>13</td>
<td>NSBA CS</td>
<td>NSBA Continuous Span Standards</td>
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SUMMARY

Beam Depth 72 in
Angle size L5x5x1/2

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<td>Stiffness check</td>
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</tr>
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<td>Angle check</td>
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<tr>
<td>Connection check</td>
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MATERIALS

Steel modulus of elasticity $E = \textbf{29,000.00} \text{ ksi}$
Beam/Girder yield strength $F_y = \textbf{58.00} \text{ ksi}$
Angle yield strength $F_{yL} = \textbf{50.00} \text{ ksi}$
Angle tension strength $F_{ut} = \textbf{58.00} \text{ ksi}$
Stiffener yield strength $F_{yS} = \textbf{50.00} \text{ ksi}$
Stiffener tension strength $F_{ut} = \textbf{65.00} \text{ ksi}$
Concrete strength $f'c = \textbf{4.50} \text{ ksi}$

Steel unit weight $\gamma_s = \textbf{490.00} \text{ pcf}$
Concrete unit weight $\gamma_c = \textbf{160.00} \text{ pcf}$

GSBTW 2.3.3.1
### SUPERSTRUCTURE

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<tr>
<td>Girder spacing</td>
<td>126.00 in</td>
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<tr>
<td>Number of girders</td>
<td>7.00</td>
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<tr>
<td>Overhang</td>
<td>48.00 in</td>
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<tr>
<td>Cross-frame spacing</td>
<td>25.00 ft</td>
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<tr>
<td>Skew</td>
<td>0.00 deg</td>
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<tr>
<td>Simply supported/continuous</td>
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<table>
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<tr>
<td>Total number of cross-frames</td>
<td>8</td>
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<tr>
<td>Drop</td>
<td>2.63 in</td>
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</table>

\[
\theta_{\text{strut}} = \tan^{-1}(\text{Drop}/S) = 0.02 \text{ rad}
\]

Out-to-out bridge width:
\[
W_{\text{deck}} = 2\times\text{OVR} + (n_g - 1)\times S = 852.00 \text{ in}
\]

#### Live load distribution limits

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<th>3.50 ft</th>
<th>≤</th>
<th>S</th>
<th>≤</th>
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<tr>
<td>Span length</td>
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<td>≤</td>
<td>L</td>
<td>≤</td>
<td>240.00 ft</td>
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<tr>
<td>Number of beams</td>
<td>4.00</td>
<td>≤</td>
<td>n_g</td>
<td>OK</td>
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<td></td>
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</table>

#### ODOT Specified limits

| Maximum frame spacing | S_{cf} | ≤ | 25.00 ft | OK |
| Maximum overhang     | OVR    | ≤ | 48.00 in | OK |
| Girder spacing       | S      | ≤ | 10.50 ft | OK |
**GIRDER**

Girder/rolled shape

<table>
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<tr>
<th>Data source</th>
<th>Girder name</th>
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<td>NSBA CS</td>
<td>CS-7.5-160</td>
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**Dimensions**

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<td>Depth</td>
<td>( d = 71.56 \text{ in} )</td>
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<tr>
<td>Top flange width</td>
<td>( b_{f1} = 18 \text{ in} )</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>( b_{f2} = 20 \text{ in} )</td>
</tr>
<tr>
<td>Top flange thickness</td>
<td>( t_{f1} = 1.56 \text{ in} )</td>
</tr>
<tr>
<td>Bottom flange thickness</td>
<td>( t_{f2} = 2 \text{ in} )</td>
</tr>
<tr>
<td>Web thickness</td>
<td>( t_w = 0.5 \text{ in} )</td>
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<tr>
<td>Web depth</td>
<td>( D = 68 \text{ in} )</td>
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**Properties**

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<tr>
<td>Weight</td>
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<tr>
<td>Area</td>
<td>( A = 102.13 \text{ in}^2 )</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>( I = 94,402 \text{ in}^4 )</td>
</tr>
<tr>
<td>Top section modulus</td>
<td>( S_{top} = 2,378 \text{ in}^3 )</td>
</tr>
<tr>
<td>Bottom section modulus</td>
<td>( S_{bot} = 2,962 \text{ in}^3 )</td>
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<tr>
<td>Top section yield moment</td>
<td>( M_y = 137,944 \text{ kip-in} )</td>
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<tr>
<td>Bottom section yield moment</td>
<td>( M_y+ = 171,802 \text{ kip-in} )</td>
</tr>
<tr>
<td>Used yield moment</td>
<td>( M_x = \min(M_y, M_y+) = 137,944 \text{ kip-in} )</td>
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<tr>
<td>Positive moment effective moment of inertia</td>
<td>( I_{eff} = 1,830 \text{ in}^4 )</td>
</tr>
<tr>
<td>Negative moment effective moment of inertia</td>
<td>( I_{eff} = 2,279 \text{ in}^4 )</td>
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<tr>
<td>Used effective moment of inertia</td>
<td>( I_{eff} = 1,830 \text{ in}^4 )</td>
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<tr>
<td>Flange centroid distance</td>
<td>( h_o = d - t_{f1}/2 - t_{f2}/2 = 70 \text{ in} )</td>
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</table>
**DECK**

Barrier assumed  

Deck effective span length  \( S_{\text{deck}} = S - \frac{b_{\text{OVR}} + S/2}{2} = 9.73 \) ft

Thickness  \( t_{\text{deck}} = \max(8.5", (L+17)*12/36) = 9.00 \) in  

Supported deck width  \( b_{\text{deck}} = \frac{S}{2} = 111.00 \) in

Overhang thickness  \( t_{\text{OVR}} = t_{\text{deck}} + 4" = 13.00 \) in

Clear overhang width  \( \text{OVR}_{\text{CL}} = \text{OVR} - \frac{b_{\text{f1}}}{2} = 39.00 \) in

Total Deck Area  \( A_{\text{deck}} = b_{\text{deck}}t_{\text{deck}} + \text{OVR}_{\text{CL}}(t_{\text{OVR}} - t_{\text{deck}}) = 1,155.00 \) in\(^2\)

Bottom of beam to deck centroid  

\[
y_{\text{deck}} = d + \frac{\frac{t_{\text{deck}}^2}{2} \left( S + \frac{b_{\text{f1}}}{2} \right) + \frac{t_{\text{OVR}}^2}{2} \text{OV}_\text{CL} R_{\text{CL}}}{A_{\text{deck}}} = 75.18 \text{ in}
\]

Total deck weight  \( w_{\text{deck}} = A_{\text{deck}} \gamma_c = 1,283.33 \) lb/ft

Combined girder + deck weight  \( w_{\text{comb}} = w + w_{\text{deck}} = 1,631 \) lb/ft

% Weight of girder  \( w / w_{\text{comb}} = 21\% \)

Overhang area  \( A_{\text{OVR}} = \text{OVR}_{\text{CL}} t_{\text{OVR}} + \frac{b_{\text{f1}}}{2} t_{\text{deck}} = 588 \) in\(^2\)

Overhang centroid eccentricity  

\[
x_{\text{OVR}} = \frac{\text{OVR}_{\text{CL}} t_{\text{OVR}} \left( \frac{b_{\text{f1}}}{2} + \frac{\text{OV}_\text{CL} R_{\text{CL}}}{2} \right) + \frac{b_{\text{f1}}^2}{2} t_{\text{slab}}}{A_{\text{OVR}}} = 27.05 \text{ in}
\]

Overhang weight  \( w_{\text{OVR}} = A_{\text{OVR}} \gamma_c = 653.33 \) lb/ft

**AASHTO Live Load distribution limits**

Slab thickness  \( 4.5 \text{ in} \leq t_s \leq 12 \text{ in} \) OK

Stiffness constant back-calculated from simplified value in AASHTO Table 4.6.2.1-3

\[
\left( \frac{K_g}{12.0Lt_s^3} \right) = 1.02
\]

\[
K_g = 1.02^{10} * 12 * L * t_s^3 = 1,706,202
\]

\[
10,000 \text{ in} \leq t_s \leq 7,000,000 \text{ in} \) OK
STIFFENER

Width
Thickness
Vertical web corner clip
Horizontal web corner clip
Weld leg
Weld thickness
Web stiffener width for stiffness

\[
\begin{align*}
\text{Width} & \quad b_s = 5.00 \text{ in} \\
\text{Thickness} & \quad t_s = 0.375 \text{ in} \\
\text{Vertical web corner clip} & \quad \text{clip}_v = 2.00 \text{ in} \\
\text{Horizontal web corner clip} & \quad \text{clip}_h = 1.00 \text{ in} \\
\text{Weld leg} & \quad \text{w}_{\text{stiff}} = \frac{5}{16} \text{ in} \\
\text{Weld thickness} & \quad t_{\text{w-stiff}} = 0.707 \times \text{w}_{\text{stiff}} = 0.22 \text{ in} \\
\text{Web stiffener width for stiffness} & \quad b_{s\text{-stiff}} = b_s = 5.00 \text{ in}
\end{align*}
\]
**CROSS-FRAME**

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<tr>
<th>Type</th>
<th>Description</th>
<th>Angle Size</th>
<th>Angle Width</th>
<th>Centroid from Face</th>
<th>Thickness</th>
<th>Gross Area</th>
<th>Radius of Gyration</th>
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<tr>
<td>Type A</td>
<td>Field Welded</td>
<td>L5x5x1/2</td>
<td>b = 5.00 in</td>
<td>y₀ = 1.42 in</td>
<td>t = 0.50 in</td>
<td>Aₓ = 4.79 in²</td>
<td>rₓ = 1.53 in</td>
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</table>

<table>
<thead>
<tr>
<th>Vertical Offset</th>
<th>Horizontal Offset</th>
</tr>
</thead>
<tbody>
<tr>
<td>OFFᵥ = 2.00 in</td>
<td>OFFᵧ = 1.00 in</td>
</tr>
</tbody>
</table>

**Strut Geometry**

Strut Angle: \( \theta_{strut} = \tan^{-1}(\text{Drop}/S) = 0.02 \text{ rad} \)

Bottom of strut left end: \( y₁ = t₁₂ + \text{OFFᵥ} = 4.00 \text{ in} \)

Top of strut left end: \( y₂ = y₁ + b_f*\cos(\theta_{strut}) + (b_s - \text{OFFᵧ})*\tan(\theta) = 9.08 \text{ in} \)

Centroid line at center of girder: \( Y_{left} = y₂ - (\text{OFFᵧ} + t_w/2)*\tan(\theta) - (b - y₀)\cos(\theta) = 5.48 \)

Bottom of strut at right end: \( y₃ = t₁₂ + \text{OFFᵥ} + \text{Drop} = 6.63 \text{ in} \)

Top of strut at right end: \( y₄ = y₃ + b\cos(\theta_{strut}) + (b_s - \text{OFFᵧ})*\tan(\theta_{strut}) = 11.71 \text{ in} \)

Centroid line at center of girder: \( Y_{right} = y₃ + (\text{OFFᵧ} + t_w/2)*\tan(\theta) + y₀\cos(\theta) = 8.07 \text{ in} \)

**AD Diagonal Geometry**

Bottom of Angle: \( y₁ = y₂(\text{strut}) + \text{OFFᵥ} = 11.08 \text{ in} \)

Top of angle: \( y₂ = d + \text{Drop} - t₁₁ - \text{OFFᵥ} = 70.63 \text{ in} \)

Left edge of angle: \( x₁ = t_w/2 + \text{OFFᵧ} = 1.25 \text{ in} \)

Right edge of angle: \( x₂ = S - x₁ = 124.75 \text{ in} \)
Angle \( \theta = \arctan\left(\frac{Y_2 - Y_1}{X_2 - X_1}\right) = 0.45 \text{ rad} \)

Left End Bottom Corner
\[ X_3 = X_1 + b \sin(\theta) = 3.42 \text{ in} \]

Centroid at end of Angle
\[ X_4 = X_1 - y_b \sin(\theta) = 2.80 \text{ in} \]
\[ Y_4 = Y_1 + y_b \cos(\theta) = 12.36 \text{ in} \]

Angle centroid line intersection with web
\[ Y_5 = Y_4 \tan(\theta) = 11.01 \text{ in} \]

Right end right corner
\[ Y_6 = Y_2 - b \cos(\theta) = 66.12 \text{ in} \]

Centroid at end of angle
\[ X_7 = X_2 - y_b \sin(\theta) = 124.13 \text{ in} \]
\[ Y_7 = Y_6 + y_b \cos(\theta) = 67.40 \text{ in} \]

Angle centroid line intersection with web
\[ Y_8 = Y_7 + (S - X_7) \tan(\theta) = 68.30 \text{ in} \]
BC Diagonal Geometry

Top of angle \[ Y_1 = d - t_{11} - \text{OFF}_v = 68.00 \text{ in} \]
Bottom of angle \[ Y_2 = Y_1 + (\text{strut} + \text{Drop} + \text{OFF}_v) = 13.71 \text{ in} \]
Left edge of angle \[ X_1 = t_{1w} / 2 + \text{OFF}_w = 1.25 \text{ in} \]
Right edge of angle \[ X_2 = S - X_1 = 124.75 \text{ in} \]
Angle \[ \theta = \text{atan}((Y_2 - Y_1)/(X_2 - X_1)) = -0.41 \text{ rad} \]
Left End Top Corner \[ X_3 = X_1 - b \sin(\theta) = 3.26 \text{ in} \]
Centroid at end of Angle \[ X_4 = X_3 + (b - y_b) \sin(\theta) = 1.82 \text{ in} \]
\[ Y_4 = Y_1 - (b - y_b) \cos(\theta) = 64.72 \text{ in} \]
Angle centroid line intersection with web \[ Y_5 = Y_4 - X_4 \tan(\theta) = 65.52 \text{ in} \]

Right end right corner \[ Y_6 = Y_2 + b \cos(\theta) = 18.28 \text{ in} \]
Centroid at end of angle \[ X_7 = X_2 + (b - y_b) \sin(\theta) = 123.31 \text{ in} \]
\[ Y_7 = Y_6 + y_b \cos(\theta) = 15.01 \text{ in} \]
Angle centroid line intersection with web \[ Y_8 = Y_7 + (S - X_7) \tan(\theta) = 13.82 \text{ in} \]

Overlap Check for applicable Girder \[ Y_1(\text{BC}) - Y_1(\text{AD}) = 56.92 \text{ in} \]
-Fails if angles would overlap.

Gap between working lines \[ 2.00 \text{ in} \]
Bottom of girder to bottom diagonal working line \[ 13.63 \text{ in} \]
Top of girder to top diagonal working line \[ 6.64 \text{ in} \]
**ANGLE GEOMETRY**

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<th></th>
<th>Left</th>
<th>Right</th>
</tr>
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<tr>
<td></td>
<td>X (in)</td>
<td>Y (in)</td>
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<tr>
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<tr>
<td>AD</td>
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<tr>
<td>BC</td>
<td>0.00</td>
<td>65.52</td>
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Truss model height

\[ h_b = Y_{BC(Left)} - Y_{AC(Left)} = 60.05 \text{ in} \]

Minimum Angle

\[ θ = \min(|θ_{AD} - θ_{AC}|, |θ_{BC} - θ_{AC}|) = 23.27 \text{ deg} \]

- Arbitrary minimum angle to ensure angles act like a truss.

Maximum angle length

\[ L_{\text{max}} = 138.41 \text{ in} \]

Angles must pass the limiting slenderness ratios for both tension and compression

Radius of gyration

\[ r_x = 1.53 \text{ in} \]

Tension slenderness ratio

\[ L_{\text{max}}/r_x = 90.47 \leq 240 \]

- Tension slenderness ratio for secondary member

Effective length factor

\[ K = 1.00 \]

- Assumes angles act like pinned-pinned

Compression slenderness ratio

\[ L_{\text{max}}K/r_x = 90.47 \leq 140 \]

- OK

\[ AASHTO 6.9.3 \]

Field Welded Cross Frame_2018 12 31.xlsx 10 of 55
CONNECTIONS

FIELD CONNECTION BOLT

Grade

<table>
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<th>A325</th>
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Diameter

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Hole type

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<thead>
<tr>
<th>Hole type</th>
<th>Oversized</th>
</tr>
</thead>
</table>

Hole diameter

<table>
<thead>
<tr>
<th>d_h</th>
<th>0.8125 in</th>
</tr>
</thead>
</table>

A_h = \( \pi d_h^2 / 4 \) = 0.307 in²

Tensile strength

<table>
<thead>
<tr>
<th>F_ub</th>
<th>120.00 ksi</th>
</tr>
</thead>
</table>

Hole reduction factor

<table>
<thead>
<tr>
<th>R_p</th>
<th>0.90</th>
</tr>
</thead>
</table>

- Allow holes punched full size

Minimum bolt tension

<table>
<thead>
<tr>
<th>P_t</th>
<th>19 kip</th>
</tr>
</thead>
</table>

Bolt Edge Distance

| e_h | 1.50 |

Minimum edge distance

<table>
<thead>
<tr>
<th>e_hmin</th>
<th>0.875 in</th>
<th>OK</th>
</tr>
</thead>
</table>

AASHTO Table 6.13.2.6.6-1

End Distances

Minimum clear end distance

<table>
<thead>
<tr>
<th>e_hbrg</th>
<th>0.625 in</th>
</tr>
</thead>
</table>

Angle edge clear distance

<table>
<thead>
<tr>
<th>L_{cl} = e_{hmin} - d_h/2</th>
<th>1.094 in</th>
<th>OK</th>
</tr>
</thead>
</table>
Angle clear distance geometry:

### Bolt Stiffener Clear Distances

<table>
<thead>
<tr>
<th>Angle</th>
<th>Side</th>
<th>(b_s) - OFF (in)</th>
<th>(H_{cs}) (in)</th>
<th>(L_{cs}) (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>Left</td>
<td>4.00</td>
<td>2.47</td>
<td>2.06</td>
</tr>
<tr>
<td>AD</td>
<td>Left</td>
<td>4.00</td>
<td>2.01</td>
<td>1.81</td>
</tr>
<tr>
<td>BC</td>
<td>Left</td>
<td>4.00</td>
<td>2.09</td>
<td>1.85</td>
</tr>
</tbody>
</table>

Where:
- \(b_s\) = Stiffener width
- \(OFF\) = Horizontal offset between web and angle
- \(e\) = Edge distance
- \(\theta\) = Diagonal angle
- \(e \sin(\theta)\) = Horizontal distance from corner of angle to bolt line at end of angle
- \(e \cos(\theta)\) = Horizontal from bolt line at end of angle to center of bolt.

\[
H_{cs} = \text{Horizontal distance from center of bolt to edge of stiffener}
\]

\[
H_{cs} = b_s - OFF - e \sin(\theta) - e \cos(\theta)
\]

\[
L_{cs} = \frac{H_{cs}}{\cos(\theta)} - \frac{d_h}{2}
\]

Change \(e\) to \((b - e)\) for BC Diagonal

Minimum stiffener clear distance

\(L_{cs} = 1.81\) in

Minimum clear distance check

Min. edge distance

\(e_{\text{hmin}} = 0.88\) in

OK
WELD
Weld leg $w = \frac{5}{16}$ in
Weld thickness $t = 0.707w = 0.22$ in
Classification strength of weld metal $F_{e70} = 70.00$ ksi

Reduced length for compression capacity

<table>
<thead>
<tr>
<th></th>
<th>$L_{comp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>119.03</td>
</tr>
<tr>
<td>AD</td>
<td>130.72</td>
</tr>
<tr>
<td>BC</td>
<td>128.63</td>
</tr>
</tbody>
</table>

$$L_{comp} = L - \frac{2\left(OFF + \frac{b_z}{2}\right)}{\cos \theta}$$

Stiffener compression length
- Stiffeners are checked for compression buckling. Length taken along the centroid line of the angle from the face of the web to the end of the angle.

$$L_{stiff} = \frac{OFF}{\cos \theta} + (b - y_b)\tan \theta$$

<table>
<thead>
<tr>
<th></th>
<th>$L_{stiff}$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>1.07</td>
</tr>
<tr>
<td>AD</td>
<td>2.73</td>
</tr>
<tr>
<td>BC</td>
<td>2.55</td>
</tr>
</tbody>
</table>

COMPILED GEOMETRY CHECKS

Geometry checks on this page OK
Check of weld lengths on analysis tab OK
Overall geometry check OK
**DESCRIPTION:**

Calculate applied loads on cross frames per GSBTW (based on ASCE 7) and AASHTO LRFD

-Load convention: → = Positive Load, ← = Negative Load.

**WIND LOADING (ERECTION):**

Girder/Beam Depth
Diaphragm/Cross Frame Spacing
Design Height Above Ground
Exposure Category

- Includes flat open country and grasslands (ASCE 7 C.3.(a))

Velocity Exposure Pressure Coefficient

<table>
<thead>
<tr>
<th>Height above ground level, z (ft)</th>
<th>Exposure</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>0.57</td>
<td>0.85</td>
<td>1.03</td>
</tr>
<tr>
<td>20</td>
<td>0.62</td>
<td>0.90</td>
<td>1.08</td>
</tr>
<tr>
<td>25</td>
<td>0.66</td>
<td>0.94</td>
<td>1.12</td>
</tr>
<tr>
<td>30</td>
<td>0.70</td>
<td>0.98</td>
<td>1.16</td>
</tr>
<tr>
<td>40</td>
<td>0.76</td>
<td>1.04</td>
<td>1.22</td>
</tr>
<tr>
<td>50</td>
<td>0.81</td>
<td>1.09</td>
<td>1.27</td>
</tr>
<tr>
<td>60</td>
<td>0.85</td>
<td>1.13</td>
<td>1.31</td>
</tr>
<tr>
<td>70</td>
<td>0.89</td>
<td>1.17</td>
<td>1.34</td>
</tr>
<tr>
<td>80</td>
<td>0.93</td>
<td>1.21</td>
<td>1.38</td>
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<tr>
<td>90</td>
<td>0.96</td>
<td>1.24</td>
<td>1.40</td>
</tr>
<tr>
<td>100</td>
<td>0.99</td>
<td>1.26</td>
<td>1.43</td>
</tr>
<tr>
<td>120</td>
<td>1.04</td>
<td>1.31</td>
<td>1.48</td>
</tr>
<tr>
<td>140</td>
<td>1.09</td>
<td>1.36</td>
<td>1.52</td>
</tr>
<tr>
<td>160</td>
<td>1.13</td>
<td>1.39</td>
<td>1.55</td>
</tr>
<tr>
<td>180</td>
<td>1.17</td>
<td>1.43</td>
<td>1.58</td>
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<tr>
<td>200</td>
<td>1.20</td>
<td>1.46</td>
<td>1.61</td>
</tr>
<tr>
<td>250</td>
<td>1.28</td>
<td>1.53</td>
<td>1.68</td>
</tr>
<tr>
<td>300</td>
<td>1.35</td>
<td>1.59</td>
<td>1.73</td>
</tr>
<tr>
<td>350</td>
<td>1.41</td>
<td>1.64</td>
<td>1.78</td>
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<tr>
<td>400</td>
<td>1.47</td>
<td>1.69</td>
<td>1.82</td>
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<tr>
<td>450</td>
<td>1.52</td>
<td>1.73</td>
<td>1.86</td>
</tr>
<tr>
<td>500</td>
<td>1.56</td>
<td>1.77</td>
<td>1.89</td>
</tr>
</tbody>
</table>

Notes:

a. Linear interpolation to intermediate values of height z is acceptable.

b. Exposure categories are defined above.

**GSBTW/ASCE**

- Girder/Beam Depth
- Diaphragm/Cross Frame Spacing
- Design Height Above Ground
- Exposure Category

| D = 71.56 in |
| Spa = 25.00 ft |
| H = 50.00 ft |
| C |

Kz = 1.09

Table C.1
Topographic Factor
- No hills, ridges, or escarpments are assumed.

\[ k_{zt} = 1.00 \]

Conservative

Wind Directionality Factor
- Conservatively ignored because GSBTW is limited to tower structures and are < 1.00.

\[ k_d = 1.00 \]

Table C.2

Basic Wind Speed

\[ V = 115.00 \text{ mph} \]

Figure C.1 (a)

Figure C.1(a)—Basic Wind Speeds for Occupancy Category II Building and Other Structures (cont.)
LOADS

Gust-Effect Factor
\[ G = 0.85 \] ASCE 7 C.6

Length/exposed height ratio
\[ B/s = \geq 45 \] For ASCE Table C.5

Clearance ratio
\[ s/h = D/H = 0.12 \]

---

**ERECTION STAGE**

Velocity Exposure Pressure Coefficient
\[ K_z = 1.09 \] Table C.1

Force Coefficient
\[ C_f = 1.95 \] Table C.5, Case A

Velocity pressure
\[ q_h = 0.00256 * K_z * K_{zt} * V^2 = 36.90 \text{ psf} \]

Gross area of solid surface
\[ A_g = D \cdot Spa = 149.09 \text{ ft}^2 \]

Force applied over cross frame spacing
\[ F_{tot} = q_h G C_f A_g = 9,119 \text{ lb} \] GSBTW (C-2)

Top & bottom lateral load
\[ F = \frac{F_{tot}}{2} = 4.56 \text{ kip} \]

---

**DECK STAGE**

- Include an additional 2' of wind height above top of girder for deck formwork, machinery.

Height allowance for deck stage
\[ H_{deck} = 2.00 \text{ ft} \]

Deck height
\[ H + 2' = 52.00 \text{ ft} \]

Velocity Exposure Pressure Coefficient
\[ K_z = 1.10 \] Table C.1

Clearance ratio
\[ s/h = (D + 2')/(H + 2') = 0.15 \]

Force Coefficient
\[ C_f = 1.95 \] Table C.5, Case A

Velocity pressure
\[ q_h = 0.00256 * K_z * K_{zt} * V^2 = 37.17 \text{ psf} \]

Top area
\[ A_{gt} = (D/2 + 2') \cdot Spa = 124.54 \text{ ft}^2 \]

Force applied to top of cross-frame
\[ F_T = q_h G C_f A_{gt} = 7,674 \text{ lb} \]

Bottom area
\[ A_{gb} = D/2 \cdot Spa = 74.54 \text{ ft}^2 \]

Force applied to top of cross-frame
\[ F_T = q_h G C_f A_{gt} = 4,593 \text{ lb} \]

Top lateral load
\[ F_{WS-T} = 7.67 \text{ kip} \]

Bottom lateral load
\[ F_{WS-B} = 4.59 \text{ kip} \]
WIND LOADING (FINAL) (AASHTO LRFD):

Wind Speed, STR III = 115.00 mph

Velocity Exposure Pressure Coefficient

Gust-Effect Factor

Drag Coefficient

\[ P_z = 2.56 \times 10^{-7} V^2 K_z G C_d \]  

AASHTO Figure 3.8.1.2.1-1

Table C3.8.1.2.1-1

Table 3.8.1.2.1-1

Table 3.8.1.2.1-2

\[ P_z = 47.97 \text{ psf} \]

Final force applied to bottom half of girder/beam

\[ F = P_z \times \frac{D}{2} \times \text{Spa} = 3,576.18 \text{ lb} \]

3.58 kip
**DEAD LOAD**

Impact % $I = 0.00$ GSBTW 2.3.3.2.2

-Impact not included because the entire 25’ of deck is not placed at once. The horizontal live load and higher concrete weight accounts for any additional loading from the concrete placement.

**ERECTION STAGE**

-No direct loading on the cross-frame from dead load during erection stage.

**DECK STAGE**

-Torsion is applied to the cross-frame as a result of the overhang deck concrete. Only the part of the overhang supported by the bracket is applied to the cross frame.

From ODOT BDM

Overhang $OVR = 48.00$ in

Platform total overhang

$OVR_{plat} = OVR + 5” + 12” + 5” = 70.00$ in

Bracket support to centerline of girder

$b_{brack} = OVR_{plat} - 16” = 54.00$ in

Width of area load applied

$b_{deck} = OVR - b_{brack}/2 = 21.00$ in

Overhang thickness

$t_{OVR} = 13.00$ in

Overhang weight

$w_{OVR} = b_{deck} * t_{OVR} * γ_c = 303.33$ lb/ft

Eccentricity

$x_{OVR} = OVR - b_{deck}/2 = 37.50$ in

Cross frame spacing

$S_{cf} = 300$ in

Girder spacing

$S = 126.00$ in

Overhang moment

$M_{OVR} = w_{OVR} x_{OVR} S_{cf} = 284.38$ kip-in

Platform weight

$p_{D} = 10.00$ psf

-Platform weight is included with $C_o$ (Construction dead load) in the GSBTW, but is included with the other dead loads for -ODOT BDM 302.2.7.2.c.E weight of the formwork used for the platform.

Platform width

Total load

Moment arm

Platform moment

Assumptions for TAEG Bracket Design Input

- $w_{plat} = 5” + 12” + 5” = 22.00$ in

ODOT BDM Assumptions for TAEG Bracket Design Input

$V_{plat} = p_{D} * w_{plat} * S_{cf} = 0.46$ kip

$x_{plat}^2 = w_{plat}/2 + OVR = 59.00$ in

$M_{plat} = V_{plat} * x_{plat} = 27.04$ kip-in
Overhang bracket weight
\[ p_{\text{brack}} = 50.00 \text{ lb} \]

Overhang bracket spacing
\[ s_{\text{brack}} = 48.00 \text{ in} \]

Overhang bracket distributed weight
\[ w_{\text{brack}} = \frac{p_{\text{brack}}}{s_{\text{brack}}} = 1.04 \text{ lb/in} \]

Overhang assumed eccentricity
\[ e_{\text{brack}} = \frac{\text{OVR}}{2} = 24.00 \text{ in} \]

Bracket moment
\[ M_{\text{brack}} = w_{\text{brack}} \times S_{\text{cf}} \times e_{\text{brack}} = 7.5 \text{ kip-in} \]

Dead load moment
\[ M_{\text{DC}} = M_{\text{OVR}} + M_{\text{plat}} + M_{\text{brack}} = 318.92 \text{ kip-in} \]

Truss height
\[ h_b = 60.05 \text{ in} \]

Top of frame lateral load
\[ F_{\text{DT}} = \frac{-M_{\text{DC}}}{h_b} = -5.31 \text{ kip} \]

Bottom of frame lateral load
\[ F_{\text{DB}} = \frac{M_{\text{DC}}}{h_b} = 5.31 \text{ kip} \]

Top of frame impact load
\[ F_{\text{Imp}} = F_{\text{DT}} = 0.00 \text{ kip} \]

Bottom of frame impact load
\[ F_{\text{IB}} = F_{\text{DB}} = 0.00 \text{ kip} \]
CONSTRUCTION LIVE LOAD  
ODOT BDM 302.2.7.2.c & GSBTW 2.3.3.2.1

Weight of equipment is assumed to be the machine wheel loads and area load are specified in ODOT BDM 302.2.7.2.c.E.6. The linear additional load from the GSBTW 2.3.3.2.1 has been removed because the machine wheel loads are based on the largest screed machine. Used for typical bridges. The area load on the walkway is for people walking along the deck. The area load on the deck is for loading from the finishing machine on the wet concrete.
All live loads are applied simultaneously.

WHEEL LOAD

Skew  
Nearest 5 degrees of skew  
Bridge out-to-out width measured perpendicular to centerline of bridge  
Centerline to centerline of screed rails

W_wheel = 4\times(2.2 \text{kip} + \max(0, L_{\text{machine}} - 36)\times0.012 \text{kip}) \times \text{Dist\%} = 10.06 \text{kip} \quad -\text{All 4 wheels combined}

Wheel load application length  
Distributed load  
Cross-frame spacing  
Applied load  
Moment arm  
Vertical load  
Cross frame moment

Wheel Load Distribution

Field Welded Cross Frame_2018 12 31.xlsm 20 of 55
AREA LOAD

- Live load is applied on the slab and walkway.

Live load on slab

\[ p_{slab} = 20.00 \text{ psf} \]

Platform total overhang

\[ OVR_{plat} = OVR + 5'' + 12'' + 5'' = 70.00 \text{ in} \]

Bracket support to centerline of girder

\[ b_{brack} = OVR_{plat} - 16'' = 54.00 \text{ in} \]

Width of area load applied

\[ b_{slab} = OVR/2 = 24.00 \text{ in} \]

- Includes area from edge of deck to halfway between bracket and centerline of girder.

Load applied on overhang

\[ w_{slab} = p_{slab} * b_{slab} = 3.33 \text{ lb/in} \]

Area load moment arm

\[ x_{slab} = 3 * b_{slab}/4 = 18.00 \text{ in} \]

Live load on walkway

\[ p_{walk} = 10.00 \text{ psf} \]

Width of area load applied

\[ b_{walk} = 16'' = 16.00 \text{ in} \]

- Includes area of platform

Load applied on overhang

\[ w_{walk} = p_{walk} * b_{walk} = 1.11 \text{ lb/in} \]

Area load moment arm

\[ x_{walk} = OVR_{plat} - b_{walk}/2 = 62.00 \text{ in} \]

Vertical load

\[ V_{area} = (w_{slab} + w_{walk}) * S_{fc} = 1.33 \text{ kip} \]

Cross frame moment

\[ M_{area} = (w_{slab} * x_{slab} + w_{walk} * x_{walk}) * S_{fc} = 38.67 \text{ kip-in} \]

LINEAR LOAD (REMOVED, SEE DISCUSSION ABOVE)

Live load

\[ w_{lin} = 0.00 \text{ lb/ft} \]

Maximum applied length

\[ L_{lin} = 240 \text{ in} \]

Applied length

\[ L_{lin-app} = \min(L_{appr}, S_{fc}) = 240 \text{ in} \]

Moment arm

\[ x_{lin} = OVR = 48 \text{ in} \]

Vertical load

\[ V_{lin} = w_{lin} * L_{lin} = 0 \text{ kip} \]

Cross frame moment

\[ M_{lin} = w_{lin} * L_{lin-app} * x_{lin} = 0 \text{ kip-in} \]

COMBINED LIVE LOAD

Total vertical load

\[ V_{LL} = V_{wheel} + V_{area} + V_{lin} = 11.39 \text{ kip} \]

Total live load moment

\[ M_{LL} = M_{wheel} + M_{area} + M_{lin} = 546.74 \text{ kip-in} \]

Truss height

\[ h_b = 60.05 \text{ in} \]

Top of frame lateral load

\[ F_{DT} = M_{DC}/h_b = -9.11 \text{ kip} \]

Bottom of frame lateral load

\[ F_{DB} = -M_{DC}/h_b = 9.11 \text{ kip} \]
HORIZONTAL LOADING

2% of Vertical load applied horizontally. Load is distributed spatially in proportion to the mass. Can be applied in either direction, but not simultaneous with wind loading.

ERECTION STAGE
- Only the weight of the girder is included

Girder weight \( w = 347.51 \text{ lb/ft} \)
Diaphragm spacing \( S_{cf} = 25.00 \text{ ft} \)

Total load
\[
2F_{Ch} = 0.02wS_{cf} = 0.17 \text{ kip}
\]

Top and bottom applied load
\( F_{Ch} = 0.09 \text{ kip} \)

DECK STAGE
- Girder weight, deck weight, and construction live load are included in the horizontal loading.
- Half of the girder weight is applied to the bottom of the cross-frame.
- All other loads are applied at the top of the cross-frame.

Deck weight \( w_{deck} = 1,283.33 \text{ lb/ft} \)

Load applied at top of cross-frame
\[
F_{ChT} = 0.02*((w + w_{deck}/2)S_{cf} + V_{LL}) = 0.73 \text{ kip}
\]

Load applied a bottom of cross-frame
\[
F_{ChB} = 0.02*w*S_{cf}/2 = 0.09 \text{ kip}
\]
GIRDER MOMENT

The strength requirements in NSBA are based on the moment applied to the girder. For a given uniform load, the maximum positive moment occurs at the center of a simple span. Similarly, the maximum negative moment comes over the center of a two-span bridge with equal spans.

- Both of these moments = 0.125wl^2 (AISC Table 3-22c & 3-23 Case 1).
- As discussed in the AISC Moments Shear and Reactions for Continuous Highway Bridges, negative moments can be increased by up to 5% due to an increase in the moment of inertia over a pier up to 50%.
- This is applied to plate girders in continuous bridges, but not wide flange or simply supported bridges.

Rolled Shape/Girder | Girder Multiplier | Span Length
--- | --- | ---
Simple span/continuous girder (SS) | m_{gird} = 1.00 | L = 160.00 ft

ERECTION STAGE

Loading consists of self weight of girder and cross-frames. Assume the weight of the cross-frames & accessories = 10% of girders. GSBTW Load combination 1 is used for the erection stage because it maximizes dead load.

Girder weight \( w_{girder} = 0.35 \text{ kip/ft} \)

Moment \( M_{f-erect} = 1.4 \times 1.1 \times m_{gird} \times w_{girder} \times L^2 \times 0.125 = 1,712.52 \text{ kip-ft} \)
DECK STAGE

Loading includes the girders, cross-frames, deck, and construction live load.

Girder weight  \( w_{\text{girder}} = 0.35 \text{ kip/ft} \)

Deck weight  \( w_{\text{deck}} = 1.28 \text{ kip/ft} \)

Dead load moment  
\[ M_{\text{D-Deck}} = (1.1w_{\text{girder}} + w_{\text{deck}})L^2 \times 0.125 = 5,329.90 \text{ kip-ft} \]

Deck moment  
\[ M_{\text{D-Deck-Only}} = (1.1w_{\text{girder}} + w_{\text{deck}})L^2 \times 0.125 = 4,517.33 \text{ kip-ft} \]

-Used for impact calculation which is based on the dead load being placed during this stage only.

Construction live load

Wheel load  \( P_{\text{wheel}} = 10.06 \text{ kip} \)

- Calculated above for construction live load

Continuous span moment  
\[ M_{\text{wheel}} = P_{\text{wheel}} \times L/(6\sqrt(3)) = 154.90 \text{ kip-ft} \]

Simple span moment  
\[ M_{\text{wheel}} = P_{\text{wheel}} \times L/4 = 402.43 \text{ kip-ft} \]

Used wheel moment  
\[ M_{\text{wheel}} = 402.43 \text{ kip-ft} \]

Area load  \( p_{\text{area}} = 20.00 \text{ psf} \)

Applied width  \( w_{\text{area}} = \text{OVR} + S/2 = 11.08 \text{ ft} \)

Distributed load  
\[ w_{\text{area}} \times p_{\text{area}} = 0.22 \text{ kip/ft} \]

Area load moment  
\[ M_{\text{area}} = w_{\text{area}} \times L^2 \times 0.125 = 709.33 \text{ kip-ft} \]

Linear load

- Conservatively collapsed to concentrated load.

Linear load total vertical force  \( V_{\text{lin}} = 0.00 \text{ kip} \)

Continuous span moment  
\[ M_{\text{L-lin}} = V_{\text{lin}} \times L/(6\sqrt(3)) = 0.00 \text{ kip-ft} \]

Simple span moment  
\[ M_{\text{L-lin}} = V_{\text{lin}} \times L/4 = 0.00 \text{ kip-ft} \]

Used linear moment  
\[ M_{\text{L-lin}} = 0.00 \text{ kip-ft} \]

Total live load moment  
\[ M_{\text{L}} = M_{\text{wheel}} + M_{\text{dist}} = 1,111.77 \text{ kip-ft} \]

Impact

% of the Dead load  
\[ M_{i} = \text{Impact\%} \times M_{\text{D-Deck}} = 0.00 \text{ kip-ft} \]

GSBTW LRFD 1  
\[ M_{1} = 1.4M_{\text{D-Deck}} = 7,461.86 \text{ kip-ft} \]

GSBTW LRFD 2  
\[ M_{1} = 1.2M_{\text{D-Deck}} + 1.6*(M_{\text{L}} + M_{i}) = 8,174.70 \text{ kip-ft} \]

Deck stage moment  
\[ M_{\text{deck}} = m_{\text{grid}} \times \max(M_{1}, M_{2}) = 8,174.70 \text{ kip-ft} \]

Check that the deck stage moment doesn’t exceed the 90% of yield moment during deck placement moment for simple spans.

110%  \( M_{y} = 12,644.84 \text{ kip-ft} \)

AASHTO Minimum Depth requirement (AASHTO Table 2.5.2.6.3-1)

Composite depth  
\[ d + t_{\text{deck}} = 80.56 \text{ in} \geq 0.04L = 76.8 \text{ in} \]

Steel depth  
\[ d = 71.56 \text{ in} \geq 0.033L = 63.36 \text{ in} \]

Field Welded Cross Frame_2018 12 31.xlsm 24 of 55
**FINAL STAGE**

Deck moment

\[ M_{D\text{-Deck}} = 5,329.90 \text{ kip-ft} \]

Barrier area

\[ A_{\text{bar}} = 4.75' \times 1.75' - 0.5 \times 4.75' \times 10.875'' = 6.16 \text{ ft}^2 \]

- Conservatively assume the weight of a SBR-2-13 57" Single Slope Concrete Bridge Railing.

![Bridge Railing Image]

Barriers

\[ N_{\text{bar}} = 2 \]

Girders

\[ n_{g} = 7 \]

Barrier weigh/girder

\[ w_{\text{bar}} = \gamma_{c} \times N_{\text{bar}} \times n_{g} = 0.28 \text{ kip/ft} \]

Barrier moment

\[ M_{D\text{-Bar}} = 901.14 \text{ kip-ft} \]

Dead load moment

\[ M_{D\text{-Final}} = M_{D\text{-Deck}} + M_{D\text{-Bar}} = 6,231.04 \text{ kip-ft} \]

Wearing surface

\[ p_{DW} = 60.00 \text{ psf} \]

Out-to-out of deck

\[ w_{\text{deck}} = 71.00 \text{ ft} \]

Distributed load

\[ w_{\text{DW}} = p_{DW} \times w_{\text{deck}} / n_{g} = 0.61 \text{ kip/ft} \]

Wearing course moment

\[ M_{\text{DW}} = w_{\text{DW}} L^2 \times 0.125 = 1,947.43 \text{ kip-ft} \]

**Live Load**

Live load tables used to calculate beam bending moments. Distribution factor taken from AASHTO.

Calculate distribution factor:

\[ \left( \frac{K_{g}}{12.0 L t_{s}^2} \right)^{0.1} = 1.02 \]

AASHTO 4.6.2.2

Girder spacing

\[ S = 10.5 \text{ ft} \]

Span length

\[ L = 160.00 \text{ ft} \]

Overhang

\[ \text{OVR} = 4.00 \text{ ft} \]

Barrier

\[ \text{BR-1-13} \]

Center of girder to face of barrier

\[ d_{e} = \text{OVR} = 2.5 \text{ ft} \]

-0' for TST-1-99, 1.5' for Other barriers.

**Interior girder**

Single lane distribution factor

\[ DF_{S} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_{g}}{12.0 L t_{s}^2} \right)^{0.1} = 0.462 \]

AASHTO 4.6.2.2b-1

Multi-lane distribution factor

\[ DF_{m} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_{g}}{12.0 L t_{s}^2} \right)^{0.1} = 0.703 \]

**Exterior girder**

Single lane distribution factor

- Lever rule. Assume TST-1-99 barrier with face at edge of overhang. First wheel is placed 2' from edge of barrier, second wheel 8' from barrier.

- 1.2 is the multiple presence factor from AASHTO Table 3.6.1.1.2-1.

\[ DF_{S\text{-ext}} = 1.2 \left( 0.5 \times \left( S + d_{e} - 2' \right) + 0.5 \times \left( S + d_{e} - 8' \right) \right) / S = 0.914 \]

Multiple lane distribution factor

\[ e = 0.77 + d_{e}/9.1 = 1.04 \]

\[ DF_{m\text{-ext}} = e \times DF_{m\text{-int}} = 0.735 \]

**Maximum exterior girder distribution factor**

\[ DF = 0.914 \]

**Moment on Simple Span**

Moment with impact taken from CalTrans Bridge Design Aids Table of Maximum Moments, Shears, and Reactions Simple Spans.

Simply supported moment

\[ M_{SS} = 5,506.00 \text{ kip-ft} \]

Distributed moment

\[ M_{LL+I} = M_{SS} \times DF = 5,034.06 \text{ kip-ft} \]
Moment on Continuous Span
Curve fit to AISC Mom. two equal span negative moments.
Two-span truck bending moment $M_{HS20} = 0.0836xL^2 + 2.8392xL + 25.261 = 2,619.69$ kip-ft

Lane load bending moment $M_{lane} = 0.64kip/ft * L^2 * 0.125 = 2,048.00$ kip-ft
Distributed moment $M_{DL} = DFx(M_{HS20} + M_{lane}) = 4,267.61$ kip-ft
Impact $M_{I} = 33%xM_{LL} = 1,408.31$ kip-ft
Live load + impact moment $M_{LL+I} = M_{LL} + M_{I} = 5,675.91$ kip-ft

Simply Supported/Simple span
Used live load + Impact moment $M_{LL+I} = 5,034.06$ kip-ft

AASHTO LRFD Load Combinations
Strength I $M_1 = 1.25M_0 + 1.5M_{DW} + 1.75M_{LL+I} = 19,519.54$ kip-ft
Final stage moment $M_{final} = m_{grid} * M_1 = 19,519.54$ kip-ft

Check that the final stage moment doesn't exceed the 110% of yield moment during final loading. moment for simple spans. $M_y = 11,495.31$ kip-ft

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<tr>
<th>P.R.</th>
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<tbody>
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BRACING STRENGTH MOMENT

Unbraced length \( L_b = 300 \text{ in} \)
Span length \( L = 1,920 \text{ in} \)
Number of span braces \( n = 8 \)
Steel modulus of elasticity \( E = 29,000 \text{ ksi} \)
Effective moment of inertia \( I_{eff} = 1,830 \text{ in}^4 \)
Moment modification factor \( C_{bb} = 1.00 \)
Flange centroid distance \( h_o = 69.78 \text{ in} \)

Erection stage
Girder moment
\[ M_{f-erect} = 20,550 \text{ kip-in} \]
Strength moment:
\[ M_{br} = \frac{0.005 L_b L M_f^2}{n E I_{eff} C_{bb}^2 h_o} = 41.05 \text{ kip-in} \]  
FHWA (14)

Deck Stage
Erection design moment
\[ M_{f-deck} = 98,096 \text{ kip-in} \]
Final stage
-Bracing only needed for continuous spans because the deck braces the compression flange for simply supported spans.
Simply Supported/Simple span SS
Erection design moment
\[ M_{f-final} = 0 \text{ kip-in} \]

\[ M_{br-final} = \frac{0.005 L_b L M_f^2}{n E I_{eff} C_{bb}^2 h_o} = 0.00 \text{ kip-in} \]  
FHWA (14)
### LOAD FACTORS AND LOAD COMBINATIONS

#### TABULATED LOADING

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<th>Stage</th>
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<th>LL</th>
<th>CH</th>
<th>WS</th>
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<td>0.00</td>
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<table>
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<th>Mbr</th>
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<td>10</td>
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</table>
REQUIRED STIFFNESS

Reduction factor \( \phi = 0.75 \)
Span length \( L = 1,920.00 \text{ in} \)
Number of span braces \( n = 8.00 \) braces
Steel modulus of elasticity \( E = 29,000.00 \text{ ksi} \)
Effective moment of inertia \( I_{\text{eff}} = 1,829.94 \text{ in}^4 \)
Moment modification factor \( C_{bb} = 1.00 \)
Skew \( \text{skew} = 0.00 \text{ deg} \)

BEAM MOMENTS

<table>
<thead>
<tr>
<th>Stage</th>
<th>Moment ( M )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>( M_{t1} = 20,550 \text{ kip-in} )</td>
</tr>
<tr>
<td>Deck Stage</td>
<td>( M_{t2} = 98,096 \text{ kip-in} )</td>
</tr>
<tr>
<td>Final stage</td>
<td>( M_{t3} = 0 \text{ kip-in} )</td>
</tr>
</tbody>
</table>

REQUIRED STIFFNESS

\[
\beta_t = \frac{2.4LM_f^2}{\phi nE I_{\text{eff}} C_{bb}^2 \cos^2 \theta} \quad \text{FHWA (13) & (19)}
\]

<table>
<thead>
<tr>
<th>Stage</th>
<th>Moment ( \beta_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>( \beta_{t1} = 6,112 \text{ kip-in/rad} )</td>
</tr>
<tr>
<td>Deck Stage</td>
<td>( \beta_{t2} = 139,262 \text{ kip-in/rad} )</td>
</tr>
<tr>
<td>Final stage</td>
<td>( \beta_{t3} = 0 \text{ kip-in/rad} )</td>
</tr>
</tbody>
</table>
Figure 9 Stiffness Formulas for Twin Girder Cross Frames [21]
Steel modulus of elasticity
\[ E = 29,000.00 \text{ ksi} \]

Girder spacing
\[ S = 126.00 \text{ in} \]

Span length
\[ L = 1,920.00 \text{ in} \]

Number of girders
\[ n_g = 7.00 \text{ girders} \]

Cross frame height
\[ h_b = 60.05 \text{ in} \]

Diagonal length
\[ L_c = 139.58 \text{ in} \]

Horizontal angle area
\[ A_h = 4.79 \text{ in}^2 \]

Horizontal angle eccentricity
\[ y_h = 1.42 \text{ in} \]

Horizontal angle thickness
\[ t_h = 0.500 \text{ in} \]

Diagonal angle area
\[ A_c = 4.79 \text{ in}^2 \]

Horizontal angle eccentricity
\[ y_c = 1.42 \text{ in} \]

Horizontal angle thickness
\[ t_c = 0.50 \text{ in} \]

Girder moment of inertia
\[ I_x = 94,402 \text{ in}^4 \]

Stiffener thickness
\[ t_s = 0.38 \text{ in} \]

**ANGLE STIFFNESS REDUCTION**

Battistini (10)

Reduction for eccentrically connected angles.

\[ R_{X_{frame}} = 1.062 - 0.087 \frac{S}{h_b} - 0.159y - 0.403t \]

Horizontal reduction
\[ R_{xh} = 0.45 \]

Horizontal effective area
\[ A_h = 2.17 \]

Diagonal reduction
\[ R_{xc} = 0.45 \]

Diagonal effective area
\[ A_c = 2.17 \]

**CROSS FRAME STIFFNESS**

Initial state
\[ \beta_b = \frac{A_c E S^2 h_b^2}{L_c^3} = 1.32E+06 \text{ kip-in/rad} -\text{Compression system} \]

Final state
\[ \beta_b = \frac{E S^2 h_b^2}{2L_c^3 + \frac{S}{A_n}} = 4.83E+05 \text{ kip-in/rad} -\text{Tension system} \]

**WEB DISTORTIONAL STIFFNESS**

Not included for full depth web stiffener with a cross frame.

\[ \beta_{sec} = \infty \]

**IN-PLANE STIFFNESS OF GIRDERS**

\[ \beta_G = \frac{24(n_g - 1)^2 S^2 E I_x}{L^3} = 7.58E+05 \text{ kip-in/rad} \]

**COMBINED STIFFNESS**

\[ \frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_G} \]

Initial state
\[ \beta_T = 4.82E+05 \text{ kip-in/rad} \]

Final state
\[ \beta_T = 2.95E+05 \text{ kip-in/rad} \]

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<thead>
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<th></th>
<th>Required</th>
<th>Provided</th>
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<tr>
<td>Erection stage</td>
<td>6.11E+03</td>
<td>≤ 4.82E+05</td>
</tr>
<tr>
<td>Deck Stage</td>
<td>1.39E+05</td>
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<tr>
<td>Final stage</td>
<td>0.00E+00</td>
<td>≤ 2.95E+05</td>
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</table>

Combined stiffness check

\[ 0.29 \]

OK
# TRUSS ANALYSIS

## GEOMETRY

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<th>Parameter</th>
<th>Value</th>
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<td>Span length</td>
<td>$126.00$ in</td>
</tr>
<tr>
<td>Truss height</td>
<td>$60.05$ in</td>
</tr>
<tr>
<td>Drop</td>
<td>$2.63$ in</td>
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## AC

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<th>Description</th>
<th>Calculation</th>
<th>Result</th>
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<tbody>
<tr>
<td>Length</td>
<td>$L_{AC} = \sqrt{S^2 + h_D^2}$</td>
<td>$126.03$ in</td>
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<tr>
<td>Angle</td>
<td>$\theta_{AC} = \text{ATAN}(h_D/S)$</td>
<td>$0.02$ rad $1.19$ degrees</td>
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## AD

<table>
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<th>Description</th>
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<th>Result</th>
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<tbody>
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<tr>
<td>Angle</td>
<td>$\theta_{AD} = \text{ATAN}((h_b + h_D)/S)$</td>
<td>$0.46$ rad $26.45$ degrees</td>
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## BC

<table>
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<td>Angle</td>
<td>$\theta_{BC} = \text{ATAN}((h_b - h_D)/S)$</td>
<td>$0.43$ rad $24.50$ degrees</td>
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**APPLIED LOADINGS**

- From Loads calculation
- FT & FB are the loads from each load combination applied to the top and bottom of the truss respectively.

<table>
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<th>Stage</th>
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<th>FT (kip)</th>
<th>FB (kip)</th>
<th>Diagonal BC</th>
<th>Diagonal AD</th>
<th>Strut AC</th>
<th>Total Force/Moment F</th>
<th>Total Force/Moment M</th>
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</tr>
<tr>
<td>9</td>
<td>Final</td>
<td>Strength III</td>
<td>0.00</td>
<td>5.01</td>
<td>0.00</td>
<td>0.00</td>
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<td>5.01</td>
<td>150.32</td>
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<tr>
<td>10</td>
<td>Final</td>
<td>Service II</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Maximum load on erection bolt from applied loads $P_{\text{bolt-f}} = 5.01$ kip
**MAX MEMBER FORCES**

### Strength

<table>
<thead>
<tr>
<th>Member</th>
<th>Erection Tension</th>
<th>Comp.</th>
<th>Deck Tension</th>
<th>Comp.</th>
<th>Final Tension</th>
<th>Comp.</th>
</tr>
</thead>
<tbody>
<tr>
<td>AD</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>BC</td>
<td>0.00</td>
<td>-5.01</td>
<td>24.30</td>
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</table>

### Service II

<table>
<thead>
<tr>
<th>Member</th>
<th>Erection</th>
<th>Deck</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>AD</td>
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<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>BC</td>
<td>0.12</td>
<td>19.89</td>
<td>0.00</td>
</tr>
<tr>
<td>AC</td>
<td>4.56</td>
<td>17.04</td>
<td>0.00</td>
</tr>
</tbody>
</table>

-During the erection stage, the cross frame behaves as the Compression system shown in FHWA Figure 9, and there is no force on the strut.

-During the deck stage, the formwork is assumed to act as a top member, and loads are distributed as shown in that system. This is conservative when calculating loads on the angles.

-Because moments shown in FHWA Figure 9 can be applied in the other direction, the loads are calculated for one direction, but the opposite is added to the applied loads when checking the angle capacity.

-The length of each individual diagonal is substituted for Lc in FHWA Figure 9 to account for the effect of the drop.

**Erection strength moment**  
\[ M_{by} = 41.05 \text{ kip-in} \]

**Applied force**  
\[ F = \frac{M_{by}}{h_b} = 0.68 \text{ kip} \]

**Spacing**  
\[ S = 126.00 \text{ in} \]

**AD Length**  
\[ L_{AD} = 140.73 \text{ in} \]

**BC Length**  
\[ L_{BC} = 138.47 \text{ in} \]

**AD Force**  
\[ F_{AD} = 0.76 \text{ kip} \]

**BC Force**  
\[ F_{BC} = 0.75 \text{ kip} \]

**Maximum force on erection bolt**  
\[ P_{bolt} = P_{bolt-f} + \max(F_{AD}, F_{BC}) = 5.77 \text{ kip} \]

**Deck strength moment**  
\[ M_{by} = 935.48 \text{ kip-in} \]

**Applied force**  
\[ F = \frac{M_{by}}{h_b} = 15.58 \text{ in} \]

**AD Length**  
\[ L_{AD} = 140.73 \text{ in} \]

**BC Length**  
\[ L_{BC} = 138.47 \text{ in} \]

**AC Length**  
\[ L_{AC} = 126.03 \text{ in} \]

**AD Force**  
\[ F_{AD} = 2F_{\text{FL}_{AD}}/S = 34.80 \text{ kip} \]

**BC Force**  
\[ F_{BC} = 2F_{\text{FL}_{BC}}/S = 34.24 \text{ kip} \]

**AC Force**  
\[ F_{AC} = -F = -15.58 \text{ kip} \]
Final strength moment \( M_{br} = 0.00 \text{ kip-in} \)

Applied force \( F = M_{br} / h_b = 0.00 \text{ in} \)

AD Length \( L_{AD} = 140.73 \text{ in} \)

BC Length \( L_{BC} = 138.47 \text{ in} \)

AC Length \( L_{AC} = 126.03 \text{ in} \)

AD Force \( F_{AD} = 2F_{AD}/S = 0.00 \text{ kip} \)

BC Force \( F_{BC} = 2F_{BC}/S = 0.00 \text{ kip} \)

AC Force \( F_{AC} = -F = 0.00 \text{ kip} \)

<table>
<thead>
<tr>
<th>Member</th>
<th>Erection Tension</th>
<th>Comp.</th>
<th>Deck Tension</th>
<th>Comp.</th>
<th>Final Tension</th>
<th>Comp.</th>
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<td>-34.80</td>
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<tr>
<td>BC</td>
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<td>-5.76</td>
<td>58.54</td>
<td>-26.07</td>
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<td>0.00</td>
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<tr>
<td>AC</td>
<td>0.00</td>
<td>-4.56</td>
<td>-23.02</td>
<td>-36.39</td>
<td>0.00</td>
<td>-5.01</td>
</tr>
</tbody>
</table>

- Final condition does not govern and is no considered further.

**Maximum top and bottom stiffener force**

- Used for checking buckling in the stiffener plate.

Top stiffener plate \( P_{stiff-top} = \min(AD, BC) = -34.80 \text{ kip} \)

- Maximum compression in either the AD or BC max member forces including bracing forces.

Bottom stiffener plate \( P_{stiff-bot} = \min(AC) = -36.39 \text{ kip} \)

- Maximum compression in the AC max member forces including bracing forces.
STIFFENER PLATE WELD

Stiffener width \( b_s = 5.00 \) in
Top flange width \( b_{f1} = 18.00 \) in
Bottom flange width \( b_{f2} = 20.00 \) in
Web depth \( D = 68.00 \) in
Web thickness \( t_w = 0.50 \) in
Vertical stiffener clip \( O_1 = 2.00 \) in
Horizontal stiffener clip \( O_2 = 1.00 \) in

Top horizontal weld length \( L_1 = \min\left(\frac{b_{f1}}{2} - \frac{t_w}{2} - O_2 - O_2, b_s - O_2\right) = 4.0000 \) in
Vertical weld length \( L_2 = D - 2O_1 = 64.0000 \) in
Bottom horizontal weld length \( L_3 = \min\left(\frac{b_{f2}}{2} - \frac{t_w}{2} - O_2 - O_2\right) = 4.0000 \) in

Total weld length \( L = L_1 + L_2 + L_3 = 72.00 \) in
Centroid from face of web \( x_1 = 0.33 \) in
Centroid from center of web \( y_1 = 0.00 \) in
Moment of inertia about centroid \( I_x = 31,093.33 \) in\(^3\)
\( I_y = 74.67 \) in\(^3\)
\( I_p = 31,168.00 \) in\(^3\)
Top leg eccentricity
\( c_{x1} = L_1 + O_1 \cdot x_1 = 5.67 \) in
\( c_{y1} = L_2/2 + O_2 \cdot y_1 = 33.00 \) in
Bottom leg eccentricity
\( c_{x3} = L_3 + O_1 \cdot x_1 = 5.67 \) in
\( c_{y3} = L_2/2 + O_2 + y_1 = 33.00 \) in

Weld group equations:
\[ x_1 = \frac{L_1 \left( O_2 + \frac{L_1}{2}\right) + L_3 \left( O_2 + \frac{L_3}{2}\right)}{L} \]
\[ y_1 = \frac{(L_1 - L_3) \left(\frac{L_2}{2} + O_1\right)}{L} \]
\[ I_x = L_1 \left(\frac{L_2}{2} + O_1 - y_1\right)^2 + \frac{L_2}{12} + L_2 y_1^2 + L_3 \left(\frac{L_2}{2} + O_1 + y_1\right)^2 \]
\[ I_y = \frac{L_1^3}{12} + L_1 \left( O_2 + L_1 - x_1\right)^2 + L_2 x_1^2 + \frac{L_3^3}{12} + L_3 \left( O_3 + L_3 - x_1\right)^2 \]

Total force \( F \) taken from truss analysis above
Total moment \( M \) taken from truss analysis above + \( M_{br} \)
### Top leg weld force

<table>
<thead>
<tr>
<th>Comb.</th>
<th>$F$ (kip)</th>
<th>$M$ (kip-in)</th>
<th>$r_{px}$ (kip/in)</th>
<th>$r_{py}$ (kip/in)</th>
<th>$r_{mx}$ (kip/in)</th>
<th>$r_{my}$ (kip/in)</th>
<th>$r_u$ (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>41.05</td>
<td>0.00</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td>41.05</td>
<td>0.01</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>9.12</td>
<td>41.05</td>
<td>0.25</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.30</td>
</tr>
<tr>
<td>4</td>
<td>0.23</td>
<td>41.05</td>
<td>0.01</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>0.00</td>
<td>1,381.97</td>
<td>0.00</td>
<td>0.00</td>
<td>1.46</td>
<td>0.25</td>
<td>1.48</td>
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<tr>
<td>6</td>
<td>-1.31</td>
<td>2,223.80</td>
<td>-0.04</td>
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<td>2.35</td>
<td>0.40</td>
<td>2.35</td>
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<tr>
<td>7</td>
<td>12.27</td>
<td>1,772.42</td>
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<tr>
<td>8</td>
<td>-1.06</td>
<td>1,990.21</td>
<td>-0.03</td>
<td>0.00</td>
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<td>0.36</td>
<td>2.11</td>
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</table>

### Bottom leg weld force

<table>
<thead>
<tr>
<th>Comb.</th>
<th>$F$ (kip)</th>
<th>$M$ (kip-in)</th>
<th>$r_{px}$ (kip/in)</th>
<th>$r_{py}$ (kip/in)</th>
<th>$r_{mx}$ (kip/in)</th>
<th>$r_{my}$ (kip/in)</th>
<th>$r_u$ (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>41.05</td>
<td>0.00</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td>2</td>
<td>0.28</td>
<td>41.05</td>
<td>0.01</td>
<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.05</td>
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<td>41.05</td>
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<td>0.00</td>
<td>0.04</td>
<td>0.01</td>
<td>0.30</td>
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<tr>
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<td>0.00</td>
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<td>2.35</td>
<td>0.40</td>
<td>2.35</td>
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<tr>
<td>7</td>
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<td>1,772.42</td>
<td>0.34</td>
<td>0.00</td>
<td>1.88</td>
<td>0.32</td>
<td>2.24</td>
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<tr>
<td>8</td>
<td>-1.06</td>
<td>1,990.21</td>
<td>-0.03</td>
<td>0.00</td>
<td>2.11</td>
<td>0.36</td>
<td>2.11</td>
</tr>
</tbody>
</table>

Maximum Weld Force

$$w_w = 2.35 \text{ kip/in}$$

Weld thickness

$$t = 0.22 \text{ in}$$

Maximum weld stress

$$w_w/2t = 5.33 \text{ ksi}$$
**ANGLE WELDS**

Minimum weld size \( L_{\text{min}} = \max(1.5" , 4*w) = 1.5 \text{ in} \)  
AASHTO 6.13.3.5

**Angle Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle width</td>
<td>5.00 in</td>
</tr>
<tr>
<td>Centroid from face</td>
<td>1.42 in</td>
</tr>
<tr>
<td>Center of angle to centroid of angle</td>
<td>1.08 in</td>
</tr>
</tbody>
</table>

**A-D DIAGONAL WELD**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle ( \theta )</td>
<td>24.45 deg</td>
</tr>
<tr>
<td>Stiffener width ( b_s )</td>
<td>5.00 in</td>
</tr>
<tr>
<td>Web-angle gap ( \text{gap} )</td>
<td>1.00 in</td>
</tr>
<tr>
<td>Horizontal overlap ( L_h )</td>
<td>4.00</td>
</tr>
<tr>
<td>Maximum ( L_3 )</td>
<td></td>
</tr>
</tbody>
</table>

**Dimensions**

- Bottom Weld Length \( L_1 \) = 2.12 in
- Top Weld Length \( L_3 \) = 4.39 in
- Front Weld Length \( L_2 \) = 5.49 in
- Front weld centroid from back of angle \( x_2 \) = 3.26 in
- Total length \( L \) = 12.01 in
- Centroid from end of angle \( x_3 \) = 2.48 in
- Centroid from middle of angle width \( y_3 \) = 0.47 in
- Moment of inertia about X Axis \( I_{xx} \) = 49.47 in^3
  \( I_{yy} \) = 18.18 in^3
- \( I_p = I_{xx} + I_{yy} = 67.64 \text{ in}^3 \)
Weld group moment of inertia equations:

\[ I_{xx} = L_1 \left( \frac{b}{2} + y_1 \right)^2 + \frac{L_2}{12} b^2 + L y_1^2 + L_3 \left( \frac{b}{2} - y_1 \right)^2 \]

\[ I_{yy} = \frac{L_1^3}{12} + L_1 \left( \frac{L_1}{2} - x_1 \right)^2 + \frac{L_2 (L_3 - L_1)^2}{12} + L_2 (X_1 - X_2)^2 + \frac{L_3^3}{12} + L_3 \left( \frac{L_3}{2} - x_1 \right)^2 \]

**Force in weld:**

Determined using a unit load along the centroid of the angle.

Force/in from direct loading

\[ r_{px} = -\frac{1}{L} = -0.083 \text{ kip/in/kip} \]

\[ r_{py} = 0 \text{ kip/in/kip} \]

Load eccentricity:

\[ e = \max(e_L - y_1, e_L + y_1) = 1.55 \text{ in} \]

e is taken as the maximum possible eccentricity because Cross-frame weld is not specific to orientation of angle.

\[ r_{mx} = \frac{1}{L} c_y, \quad r_{my} = -\frac{1}{L} c_x \]

<table>
<thead>
<tr>
<th>Location</th>
<th>cx (in)</th>
<th>cy (in)</th>
<th>rmx (kip/in/in)</th>
<th>rmy (kip/in/in)</th>
<th>ru (kip/in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Bot.-left</td>
<td>-2.48</td>
<td>-2.97</td>
<td>-0.07</td>
<td>0.06</td>
<td>0.16</td>
</tr>
<tr>
<td>2 Bot.-right</td>
<td>-0.36</td>
<td>-2.97</td>
<td>-0.07</td>
<td>0.01</td>
<td>0.15</td>
</tr>
<tr>
<td>3 Top-right</td>
<td>1.91</td>
<td>2.03</td>
<td>0.05</td>
<td>-0.04</td>
<td>0.06</td>
</tr>
<tr>
<td>4 Top-left</td>
<td>-2.48</td>
<td>2.03</td>
<td>0.05</td>
<td>0.06</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Maximum load / axial load on angle

0.162 kip/in/in
**B-C DIAGONAL WELD**

Angle \( \theta = -22.31 \text{ deg} \)

Stiffener width \( b_f = 5.00 \text{ in} \)

Web-angle gap \( \text{gap} = 1.00 \text{ in} \)

Horizontal overlap \( L_h = 4.00 \)

Maximum \( L_1 \)

Minimum weld length

Bottom Weld Length \( L_1 = 4.32 \text{ OK} \)

Top Weld Length \( L_3 = 5.40 \text{ in} \)

Front Weld Length \( L_2 = 2.27 \text{ OK} \)

Front weld centroid from back of angle \( x_2 = 3.30 \text{ in} \)

Total length \( L = 12.00 \text{ in} \)

Centroid from end of angle \( x_1 = 2.48 \text{ in} \)

Centroid from middle of angle width \( y_1 = -0.43 \text{ in} \)

Moment of inertia about X Axis
\( I_{xx} = 50.29 \text{ in}^3 \)
\( I_{yy} = 17.76 \text{ in}^3 \)
\( I_p = I_{xx} + I_{yy} = 68.06 \)

**Force in weld:**

Determined using a unit load along the centroid of the angle.

Force/in from direct loading
\( r_{px} = -1/L = -0.083 \text{ kip/in/kip} \)
\( r_{py} = 0 = 0.000 \text{ kip/in/kip} \)

Load eccentricity:
\( e = \max(eL - y_1, eL + y_1) = 1.51 \text{ in} \)

\( e \) is taken as the maximum possible eccentricity because Cross-frame weld is not specific to orientation of angle.

\[
\begin{align*}
\dot{r}_{mx} &= \frac{1ec_y}{I_p}, \\
\dot{r}_{my} &= -\frac{1ec_x}{I_p}
\end{align*}
\]

<table>
<thead>
<tr>
<th>Location</th>
<th>( cx )</th>
<th>( cy )</th>
<th>( \dot{r}_{mx} )</th>
<th>( \dot{r}_{my} )</th>
<th>( ru )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Bot.-left</td>
<td>-2.48</td>
<td>-2.07</td>
<td>-0.05</td>
<td>0.06</td>
<td>0.14</td>
</tr>
<tr>
<td>2 Bot.-right</td>
<td>1.84</td>
<td>-2.07</td>
<td>-0.05</td>
<td>-0.04</td>
<td>0.14</td>
</tr>
<tr>
<td>3 Top-right</td>
<td>-0.21</td>
<td>2.93</td>
<td>0.07</td>
<td>0.00</td>
<td>0.02</td>
</tr>
<tr>
<td>4 Top-left</td>
<td>-2.48</td>
<td>2.93</td>
<td>0.07</td>
<td>0.06</td>
<td>0.06</td>
</tr>
</tbody>
</table>

Maximum load / axial load on angle \( 0.143 \text{ kip/in/in} \)
A-C STRUT WELD

Angle \( \theta = 1.18 \) deg

Stiffener width \( bf = 5.00 \) in

Web-angle gap \( gap = 1.00 \) in

Horizontal overlap \( Lh = 4.00 \) in

Maximum \( L_3 \)

Minimum weld length

Bottom Weld Length \( L_1 = 3.90 \) OK

Top Weld Length \( L_3 = 4.00 \) OK

Front Weld Length \( L_2 = 5.00 \) in OK

Front weld centroid from back of angle \( x_2 = 3.95 \) in

Total length \( L = 12.90 \) in

Centroid from end of angle \( x_1 = 2.74 \) in

Centroid from middle of angle width \( y_1 = 0.02 \) in

Moment of inertia about X Axis \( I_{xx} = 59.78 \) in^3

\( I_{yy} = 22.22 \) in^3

\( I_p = I_{xx} + I_{yy} = 82.00 \)

Force in weld:

Determined using a unit load along the centroid of the angle.

Force/in from direct loading

\[ r_{px} = -\frac{1}{L} = -0.078 \text{ kip/in/kip} \]

\[ r_{py} = 0 = 0.000 \text{ kip/in/kip} \]

Load eccentricity:

\[ e = \max(e_L - y_1, e_L + y_1) = 1.10 \text{ in} \]

\( e \) is taken as the maximum possible eccentricity because Cross-frame weld is not specific to orientation of angle.

\[ r_{mx} = \frac{1ec_y}{I_p}, \quad r_{my} = -\frac{1ec_x}{I_p} \]

<table>
<thead>
<tr>
<th>Location</th>
<th>( cx ) in</th>
<th>( cy ) in</th>
<th>( r_{mx} ) kip/in/in</th>
<th>( r_{my} ) kip/in/in</th>
<th>( ru ) kip/in/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>-0.06</td>
<td>-0.03</td>
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<td>2.48</td>
<td>0.06</td>
<td>0.06</td>
<td>0.07</td>
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</tbody>
</table>

Maximum load / axial load on angle \( 0.155 \) kip/in/in
WELD FORCE
Member forces from Truss Analysis above.

A-D Weld force 0.162 kip/in/in
B-C Weld force 0.143 kip/in/in
A-C Weld Force 0.155 kip/in/in

<table>
<thead>
<tr>
<th>Stage</th>
<th>Spec.</th>
<th>L.C.</th>
<th>Diagonal BC</th>
<th>Diagonal AD</th>
<th>Chord AC</th>
<th>BC</th>
<th>AD</th>
<th>AC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>GSBTW</td>
<td>LRFD 1</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
</tr>
<tr>
<td></td>
<td>GSBTW</td>
<td>LRFD 2</td>
<td>-0.15</td>
<td>0.00</td>
<td>-0.14</td>
<td>-0.02</td>
<td>0.00</td>
<td>-0.02</td>
</tr>
<tr>
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<td>GSBTW</td>
<td>LRFD 9</td>
<td>-5.01</td>
<td>0.00</td>
<td>-4.56</td>
<td>-0.72</td>
<td>0.00</td>
<td>-0.70</td>
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<td>GSBTW</td>
<td>LRFD 1</td>
<td>8.17</td>
<td>0.00</td>
<td>-7.44</td>
<td>1.17</td>
<td>0.00</td>
<td>-1.15</td>
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<td>GSBTW</td>
<td>LRFD 2</td>
<td>24.30</td>
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<td>-3.22</td>
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<td>GSBTW</td>
<td>LRFD 9</td>
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<td>1.22</td>
<td>0.00</td>
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<td>LRFD</td>
<td>Service II</td>
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<td></td>
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<td></td>
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</tr>
</tbody>
</table>

Maximum Weld Force \[ w_w = 3.47 \text{ kip/in} \]
Weld thickness \[ t = 0.22 \text{ in} \]
Maximum weld stress \[ w_w/t = 15.69 \text{ ksi} \]

All weld lengths are adequate  OK
DESCRIPTION:

Check angle capacity

A  B  C  D
AD DIAGONAL

Angle Size
Modulus of Elasticity $E = \frac{L5x5x1/2}{29,000.00}$ ksi
Yield Strength $F_{yL} = 50.00$ ksi
Ultimate Strength $F_{uL} = 58.00$ ksi
Angle leg width $b = 5.00$ in
Angle Thickness $t = 0.500$ in
Angle Area (Gross) $A_g = 4.79$ in
Radius of gyration, $r_x = 1.53$ in
Angle Length $L = 138.41$ in

NONSLENDER MEMBER ELEMENTS CHECK

Slenderness ratio $b/t = 10.00$
Plate buckling coefficient $k = 0.45$

Slenderness Limit $k \times \frac{E}{F_y} = 10.84$

Nonslender $b/t \leq k\sqrt{\frac{E}{F_y}}$ OK

Angle effective length $(\frac{KL}{r})_{eff} = \begin{cases} 72 + 0.75 \frac{L}{r_x} & \text{if } \frac{L}{r_x} \leq 80 \\ 32 + 1.25 \frac{L}{r_x} & \text{if } \frac{L}{r_x} > 80 \end{cases} = 145.08$

COMPRESSION CAPACITY

Slender Element Reduction Factor, $Q$ $Q = 1.00$
Equivalent nominal yield resistance $P_o = QF_yA_g = 239.50$ kip

$P_e = \frac{\pi^2E}{(KL/r)_{eff}} \frac{2}{A_g} = 65.13$ kip

$P_n = \begin{cases} 0.658P_e & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ 0.877P_e & \text{if } \frac{P_e}{P_o} < 0.44 \end{cases} = 57.12$ kip

Resistance factor, Compression $\phi_r = 0.95$
Compressive Resistance $P_r = \phi_rP_n = 54.27$ kip
TENSION CAPACITY

Gross area \( A_g = 4.79 \, \text{in}^2 \)
Leg width \( b = 5.00 \, \text{in} \)
Thickness \( t = 0.500 \, \text{in} \)
Centroid from face \( y_b = 1.42 \, \text{in} \)
Hole diameter \( d_h = 0.8125 \, \text{in} \)
Angle yield stress \( F_{yL} = 50.00 \, \text{ksi} \)
Angle tension stress \( F_{uL} = 58.00 \, \text{ksi} \)

Yield reduction factor \( \phi_y = 0.95 \) AASHTO 6.5.4.2
Gross section yield strength \( \phi_y P_{ny} = \phi_y F_y A_g = 227.53 \, \text{kip} \) AASHTO 6.8.2.1-1

Erection Stage Net Section Fracture

A single bolt connection is not covered by AASHTO. AISC commentary D3.3 states that it is probably conservative to use the net area of the connected element.

\( U_{an} = (b - d_h) \times t = 2.09 \, \text{in}^2 \)
Hole reduction factor \( R_p = 0.90 \) AASHTO 6.8.2.1
Fracture reduction factor \( \phi_u = 0.80 \) AASHTO 6.5.4.2
Angle tension capacity \( \phi_u P_{nu} = \phi_u F_u A_n R_p U = 87.44 \, \text{kip} \) AASHTO 6.8.2.1-2
Governing tension resistance \( P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 87.44 \, \text{kip} \)

Deck Stage Net Section Fracture

Connection eccentricity, \( \bar{x} \)
\( \bar{x} = y_b = 1.42 \) AASHTO Table 6.8.2.2-1
Connection Length \( L = \min(L_1, L_2) = 2.12 \, \text{in} \)
Shear Lag Reduction Factor, \( U = 1 - \bar{x} / \text{connection length} \)
\( U = 0.33 \) AASHTO 6.8.2.2
Reduction Factor for Holes, \( R_p \)
\( R_p = 1.00 \) AASHTO 6.8.2.1

- No holes for welded connection
Angle tension capacity \( \phi_u P_{nu} = \phi_u F_u A_n R_p U = 73.43 \, \text{kip} \) AASHTO 6.8.2.1-2
Governing tension resistance \( P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 73.43 \, \text{kip} \)
BC DIAGONAL

Angle Size
Modulus of Elasticity 
Yield Strength 
Ultimate Strength 
Angle leg width 
Angle Thickness 
Angle Area (Gross) 
Radius of gyration, $r_x$ 
Angle Length

$E = 29,000.00$ ksi
$F_{yL} = 50.00$ ksi
$F_{uL} = 58.00$ ksi
$b = 5.00$ in
$t = 0.500$ in
$A_g = 4.79$ in
$r_x = 1.53$ in
$L = 136.19$ in

NONSLENDER MEMBER ELEMENTS CHECK

Slenderness ratio
Plate buckling coefficient
Slenderness Limit
Nonslender
Angle effective length

$b/t = 10.00$
$k = 0.45$
$k \frac{E}{F_y} = 10.84$
$b/t \leq k\sqrt{\frac{E}{F_y}}$
OK

(AASHTO 6.9.4.2.1)
(AASHTO Table 6.9.4.2.1-1)

(AASHTO 6.9.4.2.1-1)

(AASHTO 6.9.4.4)

COMPRESSION CAPACITY

Slender Element Reduction Factor, $Q$
Equivalent nominal yield resistance

$Q = 1.00$
$P_o = QF_yA_g = 239.50$ kip

(AASHTO Table 6.9.4.1.1-1)

(AASHTO 6.9.4.1.1)

(AASHTO 6.9.4.1.2-1)

Resistance factor, Compression
Compressive Resistance

$\phi_c = 0.95$
$P_r = \phi_cP_n = 55.65$ kip

(AASHTO 6.5.4.2)
**TENSION CAPACITY**

Gross area \( A_g = 4.79 \text{ in}^2 \)

Leg width \( b = 5.00 \text{ in} \)

Thickness \( t = 0.500 \text{ in} \)

Centroid from face \( y_b = 1.42 \text{ in} \)

Hole diameter \( d_h = 0.8125 \text{ in} \)

Angle yield stress \( F_{yL} = 50.00 \text{ ksi} \)

Angle tension stress \( F_{uL} = 58.00 \text{ ksi} \)

Yield reduction factor \( \phi_y = 0.95 \)

AASHTO 6.5.4.2

Gross section yield strength

\[
\phi_y P_{ny} = \phi_y F_y A_g = 227.53 \text{ kip}
\]  

AASHTO 6.8.2.1-1

**Erection Stage Net Section Fracture**

A single bolt connection is not covered by AASHTO. AISC commentary D3.3 states that it is probably conservative to use the net area of the connected element.

\[
UA_n = (b - d_h) * t = 2.09 \text{ in}^2
\]

Hole reduction factor \( R_p = 0.90 \)

AASHTO 6.8.2.1

Fracture reduction factor \( \phi_u = 0.80 \)

AASHTO 6.5.4.2

Angle tension capacity

\[
\phi_u P_{nu} = \phi_u F_u A_n R_p U = 87.44 \text{ kip}
\]  

AASHTO 6.8.2.1-2

Governing tension resistance

\[
P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 87.44 \text{ kip}
\]

**Deck Stage Net Section Fracture**

Connection eccentricity, \( x \)

\( x = y_b = 1.42 \)

AASHTO Table 6.8.2.2-1

Connection Length

\( L = \min(L_1, L_3) = 2.27 \text{ in} \)

Shear Lag Reduction Factor, \( U = 1 - \frac{x}{\text{connection length}} \)

\( U = 0.38 \)

AASHTO 6.8.2.2

Reduction Factor for Holes, \( R_p \)

\( R_p = 1.00 \)

AASHTO 6.8.2.1

- No holes for welded connection

Angle tension capacity

\[
\phi_u P_{nu} = \phi_u F_u A_n R_p U = 83.35 \text{ kip}
\]  

AASHTO 6.8.2.1-2

Governing tension resistance

\[
P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 83.35 \text{ kip}
\]
**ANGLE DESIGN CHECKS**

**AC STRUT**

Angle Size  
Modulus of Elasticity  
Yield Strength  
Ultimate Strength  
Angle leg width  
Angle Thickness  
Angle Area (Gross)  
Radius of gyration, x  
Angle Length  

- **L5x5x1/2**  
- \( E = 29,000.00 \) ksi  
- \( F_{yL} = 50.00 \) ksi  
- \( F_{uL} = 58.00 \) ksi  
- \( b = 5.00 \) in  
- \( t = 0.50 \) in  
- \( A_g = 4.79 \) in  
- \( r_x = 1.53 \) in  
- \( L = 119.03 \) in

**NONSLENDER MEMBER ELEMENTS CHECK**  
- Slenderness ratio \( b/t = 10.00 \)  
- Plate buckling coefficient \( k = 0.45 \)  
- Slenderness Limit \( k \frac{E}{F_y} = 10.84 \)  
- Nonslender \( b/t \leq k \sqrt{\frac{E}{F_y}} \) OK

**Angle effective length**  
\[
\left(\frac{KL}{r}\right)_{eff} = \begin{cases} 
72 + 0.75 \frac{L}{r_x} & \text{if } \frac{L}{r_x} \leq 80 \\
32 + 1.25 \frac{L}{r_x} & \text{if } \frac{L}{r_x} > 80
\end{cases}
\]
\( = 130.35 \) in

**COMPRESSION CAPACITY**  
- Slender Element Reduction Factor, \( Q = 1.00 \)  
- Equivalent nominal yield resistance \( P_o = Q F_y A_g = 239.50 \) kip  
- \( P_e = \frac{\pi^2 E}{(KL/r)_{eff}} A_g = 80.69 \) kip  
- \( P_n = \begin{cases} 
0.658 P_o P_o & \text{if } \frac{P_e}{P_o} \geq 0.44 \\
0.877 P_e & \text{if } \frac{P_e}{P_o} < 0.44
\end{cases} = 70.77 \) kip

- Resistance factor, Compression \( \phi_c = 0.95 \)  
- Compressive Resistance \( P_r = \phi_c P_n = 67.23 \) kip
**TENSION CAPACITY**

Gross area $A_g = 4.79 \text{ in}^2$

Leg width $b = 5.00 \text{ in}$

Thickness $t = 0.500 \text{ in}$

Centroid from face $y_b = 1.42 \text{ in}$

Hole diameter $d_h = 0.8125 \text{ in}$

Angle yield stress $F_{yL} = 50.00 \text{ ksi}$

Angle tension stress $F_{uL} = 58.00 \text{ ksi}$

Yield reduction factor $\phi_y = 0.95$ AASHTO 6.5.4.2

Gross section yield strength $\phi_y P_{ny} = \phi_y F_y A_g = 227.53 \text{ kip}$ AASHTO 6.8.2.1-1

**Erection Stage Net Section Fracture**

A single bolt connection is not covered by AASHTO. AISC commentary D3.3 states that it is probably conservative to use the net area of the connected element.

$$UA_n = (b - d_h) \times t = 2.09 \text{ in}^2$$

Hole reduction factor $R_p = 0.90$ AASHTO 6.8.2.1

Fracture reduction factor $\phi_u = 0.80$ AASHTO 6.5.4.2

Angle tension capacity $\phi_u P_{nu} = \phi_u F_u A_n R_p U = 87.44 \text{ kip}$ AASHTO 6.8.2.1-2

Governing tension resistance $P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 87.44 \text{ kip}$

**Deck Stage Net Section Fracture**

Connection eccentricity, $\bar{x} = y_b = 1.42$ AASHTO Table 6.8.2.2-1

Connection Length $L = \min(L_1, L_2) = 3.90 \text{ in}$

Shear Lag Reduction Factor, $U = 1 - \frac{x}{connection \ length} = 0.64$ AASHTO 6.8.2.2

Reduction Factor for Holes, $R_p = 1.00$ AASHTO 6.8.2.1

Shear Lag Reduction Factor

$$U = 1 - \frac{x}{L} = 0.64$$

Reduction Factor for Holes, $R_p = 1.00$

Angle tension capacity $\phi_u P_{nu} = \phi_u F_u A_n R_p U = 141.29 \text{ kip}$ AASHTO 6.8.2.1-2

Governing tension resistance $P_r = \min(\phi_y P_{ny}, \phi_u P_{nu}) = 141.29 \text{ kip}$
## Design Checks

### AD DIAGONAL

<table>
<thead>
<tr>
<th>Stage</th>
<th>Tens./Com.</th>
<th>Load</th>
<th>Cap</th>
<th>P.R.</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>T</td>
<td>0.76 kip</td>
<td>≤ 87.44 kip</td>
<td>0.01</td>
<td>OK</td>
</tr>
<tr>
<td>Erection</td>
<td>C</td>
<td>-0.76 kip</td>
<td>≥ -54.27 kip</td>
<td>0.01</td>
<td>OK</td>
</tr>
<tr>
<td>Deck</td>
<td>T</td>
<td>34.80 kip</td>
<td>≤ 73.43 kip</td>
<td>0.47</td>
<td>OK</td>
</tr>
<tr>
<td>Deck</td>
<td>C</td>
<td>-34.80 kip</td>
<td>≥ -54.27 kip</td>
<td>0.64</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>T</td>
<td>0.00 kip</td>
<td>≤ 73.43 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>C</td>
<td>0.00 kip</td>
<td>≥ -54.27 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
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</table>

### BC DIAGONAL

<table>
<thead>
<tr>
<th>Stage</th>
<th>Tens./Com.</th>
<th>Load</th>
<th>Cap</th>
<th>P.R.</th>
<th>Check</th>
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<tbody>
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<td>T</td>
<td>0.75 kip</td>
<td>≤ 87.44 kip</td>
<td>0.01</td>
<td>OK</td>
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<tr>
<td>Erection</td>
<td>C</td>
<td>-5.76 kip</td>
<td>≥ -55.65 kip</td>
<td>0.10</td>
<td>OK</td>
</tr>
<tr>
<td>Deck</td>
<td>T</td>
<td>58.54 kip</td>
<td>≤ 83.35 kip</td>
<td>0.70</td>
<td>OK</td>
</tr>
<tr>
<td>Deck</td>
<td>C</td>
<td>-26.07 kip</td>
<td>≥ -55.65 kip</td>
<td>0.47</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>T</td>
<td>0.00 kip</td>
<td>≤ 83.35 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>C</td>
<td>0.00 kip</td>
<td>≥ -55.65 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
</tbody>
</table>

### AC STRUT

<table>
<thead>
<tr>
<th>Stage</th>
<th>Tens./Com.</th>
<th>Load</th>
<th>Cap</th>
<th>P.R.</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erection</td>
<td>T</td>
<td>0.00 kip</td>
<td>≤ 87.44 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Erection</td>
<td>C</td>
<td>-4.56 kip</td>
<td>≥ -67.23 kip</td>
<td>0.07</td>
<td>OK</td>
</tr>
<tr>
<td>Deck</td>
<td>T</td>
<td>-23.02 kip</td>
<td>≤ 141.29 kip</td>
<td>-0.16</td>
<td>OK</td>
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<tr>
<td>Deck</td>
<td>C</td>
<td>-36.39 kip</td>
<td>≥ -67.23 kip</td>
<td>0.54</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>T</td>
<td>0.00 kip</td>
<td>≤ 141.29 kip</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Final</td>
<td>C</td>
<td>-5.01 kip</td>
<td>≥ -67.23 kip</td>
<td>0.07</td>
<td>OK</td>
</tr>
</tbody>
</table>

Overall angle design 0.70 OK
**DESCRIPTION:**

Calculate design checks for cross frame connections

**BOLT RESISTANCE**

**Shear Resistance of Bolts (AASHTO 6.13.2.7)**

<table>
<thead>
<tr>
<th>THREADS EXCLUDED FROM SHEAR PLANE</th>
<th>FALSE</th>
</tr>
</thead>
</table>

- **Bolt area** \( A_b = 0.31 \text{ in}^2 \)
- **Bolt tensile strength** \( F_{ub} = 120.00 \text{ ksi} \)
- **Number of shear planes** \( N_s = 1.00 \)
- **Resistance factor** \( \phi_s = 0.80 \)

\[
\phi_s R_n = \phi_s \begin{cases} 
0.48 A_b F_{ub} N_s & \text{if Threads excluded from shear plane} \\
0.38 A_b F_{ub} N_s & \text{if Threads included in shear plane}
\end{cases}
\]

\[
\phi_s R_n = 11.19 \text{ kip/bolt} \quad \text{AASHTO 6.13.2.7-1 & 6.13.2.7-2}
\]

**SLIP RESISTANCE**

<table>
<thead>
<tr>
<th>NUMBER OF SLIP PLANES PER BOLT</th>
<th>1.00</th>
</tr>
</thead>
</table>

- **Minimum required bolt tension** \( P_t = 19.00 \text{ kip} \)
- **Hole size factor** \( K_h = 0.85 \)
  - For oversized hole
- **Surface condition factor** \( K_s = 0.33 \)
  - Minimum of class A, B, C

\[
R_n = K_h K_s N_s P_t = 5.33 \text{ kip} \quad \text{AASHTO 6.13.2.8-1}
\]

**BEARING RESISTANCE**

- **Bolt diameter** \( d = 0.625 \text{ in} \)

**Bearing on Angle**

<table>
<thead>
<tr>
<th>THICKNESS</th>
<th>0.500 \text{ in}</th>
</tr>
</thead>
</table>

- **Tensile strength of angle** \( F_u = 58.00 \text{ ksi} \)
- **Clear edge distance** \( L_c = 1.094 \text{ in} \)

\[
R_n = \min(2.4 d t F_u, 1.2 L_c F_u) = 43.50 \text{ kip} \quad \text{AASHTO 6.13.2.9-1}
\]

**Bearing on Stiffener**

<table>
<thead>
<tr>
<th>THICKNESS</th>
<th>0.375 \text{ in}</th>
</tr>
</thead>
</table>

- **Tensile strength of stiffener** \( F_u = 65.00 \text{ ksi} \)
- **Clear edge distance** \( L_c = 1.806 \text{ in} \)

\[
R_n = \min(2.4 d t F_u, 1.2 L_c F_u) = 36.56 \text{ kip} \quad \text{AASHTO 6.13.2.9-1}
\]

**Resistance factor, Bearing** \( \phi_{bb} = 0.80 \)

**Bearing Resistance, Bolt Holes**

\[
R_r = \phi_{bb} \min(R_{nL}, R_n) = 29.25 \text{ kip}
\]
**BLOCK SHEAR RESISTANCE**

Reduction factor for holes

\[ R_p = 0.90 \]

Angle tensile strength

\[ F_{uL} = 58.00 \text{ ksi} \]

Angle yield strength

\[ F_{yL} = 50.00 \text{ ksi} \]

Non-uniform reduction factor

\[ U_{bs} = 0.50 \]

-Stress is non-uniform since angle is loaded through one leg only.

Bolt edge distance

\[ e_h = 1.50 \text{ in} \]

Angle thickness

\[ t = 0.500 \text{ in} \]

Hole diameter

\[ d_h = 0.8125 \text{ in} \]

Shear gross area

\[ A_{vg} = e_h \times t = 0.75 \text{ in}^2 \]

Shear net area

\[ A_{vn} = A_{vg} - d_h/2 \times t = 0.55 \text{ in}^2 \]

Tension gross area

\[ A_{tg} = e_h \times t = 0.75 \text{ in}^2 \]

Tension net area

\[ A_{tn} = A_{tg} - d_h/2 \times t = 0.55 \text{ in}^2 \]

Resistance factor

\[ \phi_{bs} = 0.80 \]

Block shear resistance

\[ R_r = \phi_{bs} R_p \min\left(0.58F_{uL}A_{vn}, 0.58F_{yL}A_{vn}, 0.58F_{uL}A_{tn}, 0.58F_{yL}A_{tn}\right) = 24.66 \text{ kip} \]

**WELD RESISTANCE**

Classification strength of weld metal

\[ F_{e70} = 70 \text{ ksi} \]

\[ \phi_{e2} = 0.80 \]

\[ R_s = 0.6\phi_{e2}F_{e70} = 33.60 \text{ ksi} \]

**BLOCK SHEAR RESISTANCE**

-Block shear rupture in the stiffener plate.

Thickness

\[ t_s = 0.375 \text{ in} \]

Hole reduction factor

\[ R_p = 1 \]

Stiffener tensile strength

\[ F_{us} = 65.00 \text{ ksi} \]

Stiffener yield strength

\[ F_{ys} = 50.00 \text{ ksi} \]

Non-uniform reduction factor

\[ U_{bs} = 0.50 \]

-Stress is non-uniform since angle is loaded through one leg only.

-Net area is the same as the gross area.

**A-D Diagonal weld**

Shear length

\[ L_v = L_1 + L_3 = 6.51 \text{ in} \]

Tension length

\[ L_t = b = 5.00 \text{ in} \]

Shear area

\[ A_{vg} = A_{vn} = L_v t_s = 2.44 \text{ in}^2 \]

Tension area

\[ A_{tg} = A_{tn} = L_t t_s = 1.88 \text{ in}^2 \]

Resistance factor

\[ \phi_{bs} = 0.80 \]

Block shear resistance

\[ R_r = \phi_{bs} R_p \min\left(0.58F_{uL}A_{vn}, 0.58F_{yL}A_{vn}, 0.58F_{uL}A_{tn}, 0.58F_{yL}A_{tn}\right) = 105.43 \text{ kip} \]
**B-C Diagonal weld**

Shear length \( L_v = L_1 + L_3 = 6.60 \text{ in} \)

Tension length \( L_t = b = 5.00 \text{ in} \)

Shear area \( A_{vg} = A_{vn} = L_v t_s = 2.47 \text{ in}^2 \)

Tension area \( A_{tg} = A_{tn} = L_t t_s = 1.88 \text{ in}^2 \)

Resistance factor \( \phi_{bs} = 0.80 \)

Block shear resistance

\[
R_r = \phi_{bs} R_p \min(0.58F_u A_{vn}, 0.58F_y A_v + U_{bs} F_u A_{tn})
\]

\[= 106.13 \text{ kip} \]

**A-C Strut Weld**

Shear length \( L_v = L_1 + L_3 = 7.90 \text{ in} \)

Tension length \( L_t = b = 5.00 \text{ in} \)

Shear area \( A_{vg} = A_{vn} = L_v t_s = 2.96 \text{ in}^2 \)

Tension area \( A_{tg} = A_{tn} = L_t t_s = 1.88 \text{ in}^2 \)

Resistance factor \( \phi_{bs} = 0.80 \)

Block shear resistance

\[
R_r = \phi_{bs} R_p \min(0.58F_u A_{vn}, 0.58F_y A_v + U_{bs} F_u A_{tn})
\]

\[= 117.47 \text{ kip} \]
CONNECTION CHECKS

CONNECTED ELEMENTS
- Check that the stiffener plate does not buckle at either the top diagonal or the bottom diagonal.

Plate yield stress \( F_y = 50.00 \text{ ksi} \)
Steel modulus of elasticity \( E = 29,000.00 \text{ ksi} \)
Reduction factor \( \phi = 0.90 \) AISC E1

TOP OF STIFFENER PLATE
- Maximum top angle compression force \( P_{\text{top-comp}} = -34.80 \text{ kip} \)
  - Taken as the maximum compression force on either diagonal.
- Maximum length \( L_{\text{stiff}} = 6.00 \text{ in} \) - Conservatively assumed 6"
- Stiffener thickness \( t_s = 0.375 \text{ in} \)
- Minimum weld length \( L_{\text{min}} = 2.12 \text{ in} \) - Minimum L1 or L3 weld length on the diagonals.

Width (Whitmore section) \( W = b + 2*L_{\text{min}}/\sqrt{3} = 7.45 \text{ in} \)
Stiffener area \( A_s = W*t_s = 2.79 \text{ in}^2 \)
Stiffener moment of inertia \( I_s = Wt_s^3/12 = 0.03 \text{ in}^4 \)
Radius of gyration \( r_s = \sqrt{I_s/A_s} = 0.11 \)
Effective length factor \( K = 0.65 \) AISC Table C-C2.2
- Assume fixed both at the web and angle. Assumed fixed in translation by the rest of the stiffener web.

Slenderness ratio \( KL_{\text{stiff}}/r_s = 36.03 \)
Stiffener is slender \( \text{TRUE} \)

Non-slender compression capacity \( \phi F_y A_g = 125.70 \text{ kip} \) AISC J4-6

Slender compression capacity
Euler buckling stress \( F_e = \pi^2 E/(KL/r)^2 = 220.52 \text{ ksi} \)
Critical buckling stress
\[
F_{cr} = \begin{cases} 
0.658 F_y & \text{if } F_e/F_y \geq 0.44 \\
0.877 F_e & \text{if } F_e/F_y < 0.44 
\end{cases}
\]
\( F_{cr} = 45.47 \text{ ksi} \) AISC E3-2 & E3-3

\( \phi F_{cr} A_g = 114.32 \text{ kip} \) AISC E3-1

Used compression capacity \( P_n = 114.32 \text{ kip} \)
**BOTTOM OF STIFFENER PLATE**

Maximum strut compression force

\[ P_{\text{top-comp}} = -36.39 \text{ kip} \]

-Taken as the maximum compression force on either diagonal.

Maximum length

\[ L_{\text{stiff}} = 6.00 \text{ in} \]

Stiffener thickness

\[ t_s = 0.375 \text{ in} \]

Minimum weld length

\[ L_{\text{min}} = 3.90 \text{ in} \]

-Minimum L1 or L3 weld length on the diagonals.

Width (Whitmore section)

\[ W = b + 2 \times L_{\text{min}} / \sqrt{3} = 9.50 \text{ in} \]

Stiffener area

\[ A_s = W 	imes t_s = 3.56 \text{ in}^2 \]

Stiffener moment of inertia

\[ I_s = W t_s^3 / 12 = 0.04 \text{ in}^4 \]

Radius of gyration

\[ r_s = \sqrt{I_s / A_s} = 0.11 \]

Effective length factor

\[ K = 0.65 \]

Assume fixed both at the web and angle. Assumed fixed in translation by the rest of the stiffener web.

Slenderness ratio

\[ K_{\text{stiff}} / r_s = 36.03 \]

Stiffener is slender

TRUE

**Non-slender compression capacity**

\[ \phi F_y A_g = 160.33 \text{ kip} \]

AISC J4-6

**Slender compression capacity**

Euler buckling stress

\[ F_e = \pi^2 E / (K L / r)^2 = 220.52 \text{ ksi} \]

Critical buckling stress

\[ F_{cr} = \begin{cases} F_y & \text{if } F_e \geq 0.44 \\ 0.877 F_e & \text{if } F_e < 0.44 \end{cases} \]

\[ = 45.47 \text{ ksi} \]

AISC E3-2 & E3-3

\[ \phi F_{cr} A_g = 145.81 \text{ kip} \]

AISC E3-1

Used compression capacity

\[ P_n = 145.81 \text{ kip} \]

**DESIGN CHECKS**

Note: Resistances are per bolt

<table>
<thead>
<tr>
<th>Loading</th>
<th>Required</th>
<th>Provided</th>
<th>P.R.</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear of Bolts</td>
<td>5.77 kip</td>
<td>\leq 11.19 kip</td>
<td>0.52</td>
<td>OK</td>
</tr>
<tr>
<td>Slip resistance</td>
<td>4.56 kip</td>
<td>\leq 5.33 kip</td>
<td>0.86</td>
<td>OK</td>
</tr>
<tr>
<td>Bearing at Bolt Holes</td>
<td>5.77 kip</td>
<td>\leq 29.25 kip</td>
<td>0.20</td>
<td>OK</td>
</tr>
<tr>
<td>Block shear rupture</td>
<td>5.77 kip</td>
<td>\leq 24.66 kip</td>
<td>0.23</td>
<td>OK</td>
</tr>
<tr>
<td>Stiffener shear rupture</td>
<td>5.33 ksi</td>
<td>\leq 33.60 ksi</td>
<td>0.16</td>
<td>OK</td>
</tr>
<tr>
<td>Fillet Welds</td>
<td>15.69 ksi</td>
<td>\leq 33.60 ksi</td>
<td>0.47</td>
<td>OK</td>
</tr>
<tr>
<td>A-D Diagonal Stiffener BSR</td>
<td>34.80 kip</td>
<td>\leq 105.43 ksi</td>
<td>0.33</td>
<td>OK</td>
</tr>
<tr>
<td>B-C Diagonal Stiffener BSR</td>
<td>58.54 kip</td>
<td>\leq 105.43 ksi</td>
<td>0.56</td>
<td>OK</td>
</tr>
<tr>
<td>A-C Strut Stiffener BSR</td>
<td>0.00 kip</td>
<td>\leq 117.47 ksi</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Top of stiffener plate compression</td>
<td>34.80 kip</td>
<td>\leq 114.32 kip</td>
<td>0.30</td>
<td>OK</td>
</tr>
<tr>
<td>Bot. of stiffener plate compression</td>
<td>36.39 ksi</td>
<td>\leq 145.81 kip</td>
<td>0.25</td>
<td>OK</td>
</tr>
</tbody>
</table>

Overall connection design

0.86 OK