T·Y·LIN INTERNATIONAL

Waitsfield Bridge No. 177
Contract Modified Deck Design and Ancillary Elements

Waitsfield, VT

Waitsfield BRF 013-4(39)

Project Design Calculations

February 3, 2016

Prepared by

T·Y·LIN INTERNATIONAL
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1. Project Description

Waitsfield Bridge No. 177 is an existing 2-span continuous rolled beam bridge constructed in 1938. A design to replace the bridge with a new single span steel beam bridge was prepared by McFarland Johnson for the Vermont Agency of Transportation (VTrans). This new bridge is a 172 ft. simple span steel I-girder structure with a composite concrete deck. This is a parallel skew bridge with skew angle of 45 degrees.

The Contractor A.L. St. Onge successfully bid the construction contract of this bridge, as advertised by VTrans. After bids were opened, the Contractor recognized an alternate construction method that could save the State money while still meeting all other project goals.

T.Y. Lin International (TYLI) was tasked by the Contractor to provide the design for an alternate deck using partial depth prestressed precast concrete panels. The calculations provided herein are only for the design of the partial depth prestressed precast concrete panels and for ancillary items that required slight modification to accommodate the changed deck thickness. The travel way consists of two 11 ft. lanes and 5 ft. shoulders on a relatively flat vertical profile. The proposed design will include an 8.5 in. concrete deck and a 0.5 in integral concrete wearing surface, and allow for a future 3 in bituminous wearing surface.

2. QC Statements of Concurrence

The following forms indicate design calculations provided herein have been checked to provide a safe, buildable, biddable project. Originator and Checker initials can also be found throughout this report.

WAITSFIELD VI

411847.00

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Deck Design and Corresponding Plans

I, Josh Olund (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____ 12/18/15
(Signature) (Date)

Checker: _____ 12/18/15
(Signature) (Date)

Additional Notes: Does not include bar details

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Deck Bar Details

I, Ben Toothaker (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____
(Signature) (Date)

Checker: _____
(Signature) (Date)

Additional Notes: _____

Watsfield VE

411847.00

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Dead load deflection and camber calculations

I, Tim Poulin (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____ 12/16/15
(Signature) (Date)

Checker: _____ 12/16/15
(Signature) (Date)

Additional Notes: _____

WAITSFIELD VE

4118+7.00

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: End Crossframe Design and Corresponding Plans

I, Tim Poulin (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____ 12/15/15
(Signature) (Date)

Checker: _____ 12/16/15
(Signature) (Date)

Additional Notes: _____

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Pedestal Elevations and Abutment 1 Backwall Elevations

I, Tim Poulin (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: Van Vhan 12/16/15.
(Signature) (Date)

Checker: 12/16/15
(Signature) (Date)

Additional Notes:

WHITFIELD VE

411847.00

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Bearing Modifications

I, JOSH OLUND (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____ 12/16/15
(Signature) (Date)

Checker: _____ 12/16/15
(Signature) (Date)

Additional Notes: _____

WAITSFIELD VE

411847.00

ATTACHMENT 1
DESIGN CHECK
STATEMENT OF CONCURRENCE

Plan Sheets or Items Reviewed: Original Plan Modifications

I, Tim Paulin (Checker), confirm that the designs and requirements noted in the documents listed above were verified in accordance with the procedures discussed in this Manual and meet the appropriate standards as outlined in the Contract and Project Scope of Work. All comments/questions proposed to the Designer during the Design Check process regarding these items are reconciled and appropriately accommodated within the documents being submitted to the Agency. By signing below, I acknowledge that I agree with the appropriateness of the design shown in the plan set submitted to the Agency.

Originator: _____ 12/10/15
(Signature) (Date)

Checker: _____ 12/18/15
(Signature) (Date)

Additional Notes: _____

3. Bridge Design

3a. Superstructure Deck Design

The partial depth precast prestressed concrete deck was designed using AASHTO LRFD Bridge 6th Edition with the 2013 interims. The deck consists of 80 partial depth precast prestressed concrete deck panels 3.5 in. deep, with a 5.5 in. cast in place concrete overlay. The precast panels were assumed to carry all the interior positive moment due to dead load, prior to curing of the cast in place overlay. The composite section was used to design the final loading conditions for positive bending between girders (prestressed section) and the negative moments above the girder lines (cast in-place section).

The precast prestressed concrete deck panel's geometry was specified based on the Precast/Prestressed Concrete Institute New England Region (PCINE) guidelines. The deck was designed with a 0.5 in. integral concrete wearing surface along with considering a future 3" bituminous wearing surface. Due to the geometry of the bridge and precast panel widths, the end of the bridge deck was treated as a conventional full depth cast in place concrete deck.

Partial Depth Precast Panel Design & Analysis

Notes/Assumptions:

1) Treat panel as fully prestressed through deck curing
2) Use Gross Section Properties
3.) Ref: AASHTO LRFD 6th Edition with 2013 interims

Properties:

$f'_{ci} := 4.0\text{ksi}$ — Concrete Strength at Release

$f'_c := 5.0\text{ksi}$ — Concrete Strength at Final

$f'_{cip} := 4.0\text{ksi}$ — Cast in Place Concrete Strength (HPC A Concrete)

$E_{ci} := 2500\text{ksi}\left(\frac{f'_{ci}}{\text{ksi}}\right)^{\frac{1}{3}} = 3968.5\,\text{ksi}$ — Modulus of Elasticity of Concrete at Release [AASHTO C5.4.2.4-1]

$E_c := 2500\text{ksi}\left(\frac{f'_c}{\text{ksi}}\right)^{\frac{1}{3}} = 4275\,\text{ksi}$ — Modulus of Elasticity of Concrete at Final [AASHTO C5.4.2.4-1]

$E_{cip} := 2500\text{ksi}\left(\frac{f'_{cip}}{\text{ksi}}\right)^{\frac{1}{3}} = 3969\cdot\text{ksi}$ — Modulus of Elasticity of Cast in Place Concrete [AASHTO C5.4.2.4-1]

$\gamma_{conc} := 150\text{pcf}$ — Unit weight of Concrete

$\gamma_{bit} := 147\text{pcf}$ — Unit weight of bituminous wearing surface

$A_{ps1} := 0.085\text{in}^2$ — Area of Single Prestressing Strand (3/8" nominal)

$N_{ps} := 16$ — Number of Strands per Panel

$f_{pu} := 270\text{ksi}$ — Ultimate Strength of tendon

$E_{ps} := 28500\text{ksi}$ — Modulus of Elasticity of Tendons

$f_{pj} := 0.75\cdot f_{pu} = 202.5\,\text{ksi}$ — Stess of Tendon at Jacking

$f_{py} := 0.90 f_{pu} = 243\,\text{ksi}$ — Yield Stress of Tendon (Low Relaxation)

$D_{bar} := 0.375\text{in}$ $\quad S_{bar} := 6\text{in}$ — Reinforcing Bar Diameter and Spacing (#3's per PCINE Precast Deck Panel Guidelines)

$A_{bar} := \pi D_{bar}^2 \div (4) = 0.11\cdot\text{in}^2$ — Reinforcing Bar Area

$F_y := 60\text{ksi}$ — Yield Stress of Mild Reinforcing Steel

$E_s := 29000\text{ksi}$ — Modulus of Elasticity of Steel

$f_r := 0.24\sqrt{f'_c\cdot\text{ksi}} = 0.54\cdot\text{ksi}$ — Rupture Modulus for determining Cracking moment [AASHTO 5.4.2.6]

Geometry:

$S_{gird} := 7ft + 6in$ — *Girder Spacing, Normal to Girders, Max*

$b_f := 20in$ — *Width of Top Flange*

$S := S_{gird} - b_f + 2 \cdot 4in = 78\,in$ — *Panel Span Width (Edge to Edge)*

$P := 3.5in$ — *Panel Thickness*

$T := 8.5in$ — *Overall Thickness of Deck, structural*

$T_{int} := 0.5in$ — *Thickness of integral wearing surface*

$T_{wear.fut} := 3in$ — *Thickness of future bituminous wearing surface*

$W := 8ft$ — *Panel Width*

$I_x := \frac{W \cdot P^3}{12} = 343\,in^4$ — *Moment of Inertia of Panel*

$S_x := \frac{2 \cdot I_x}{P} = 196 \cdot in^3$ — *Section Modulus of Panel*

Loads & Moments:

Dead Loads:

$DC1 := \gamma_{conc} \cdot W \cdot P = 350 \cdot plf$ — *Uniform Load - Self Weight*

$DC2 := \gamma_{conc} \cdot (T - P) \cdot W = 500 \cdot plf$ — *Uniform Load due to Additional Concrete*

$DC_{int} := \gamma_{conc} \cdot W \cdot T_{int} = 50 \cdot plf$ — *Uniform load due to Integral Wearing Surface*

$DW_{future} := \gamma_{bit} \cdot T_{wear.fut} \cdot W = 294 \cdot plf$ — *Uniform load due to Future Wearing Surface*

$M_{DC1} := 0.125 DC1 \cdot S^2 = 22.18 \cdot kip \cdot in$ — *Max Moment due to Self Weight*

$M_{DC2} := 0.125 DC2 \cdot S^2 = 31.69 \cdot kip \cdot in$ — *Max Moment due to Additional Concrete*

$M_{DC.int} := 0.125 \cdot DC_{int} \cdot S^2 = 3.17 \cdot kip \cdot in$ — *Max Moment due to Integral Wearing Surface*

$M_{DW.fut} := 0.125 \cdot DW_{future} \cdot S^2 = 18.63 \cdot kip \cdot in$ — *Max Moment due to Integral Wearing Surface*

Live Loads:

$LL := 50psf$ — *Construction Live Load Allowance (VTrans SDM 3.12.1)*

$M_{LL} := 0.125 \cdot LL \cdot W \cdot S^2 = 25.35 \cdot kip \cdot in$ — *Moment due to construction live load*

HL-93 Live load moments are presented later.

Pre-Stress Losses:

Use approximate losses, AASHTO 5.9.5.3

$ecc := 0 \cdot in$ — *Tendon eccentricity at critical section*

$M_o := M_{DC1} = 22.18 \cdot kip \cdot in$ — *Self-weight moment at time of stressing*

$RH := 75\%$ — *Assumed relative humidity*

Elastic Shortening:

$$\Delta f_{pES} := \frac{\left(N_{ps} \cdot A_{ps1} \cdot f_{pj} \cdot I_x\right)}{N_{ps} \cdot A_{ps1} \cdot I_x + P \cdot W \cdot I_x \cdot \frac{E_{ci}}{E_{ps}}} = 5.72 \cdot ksi$$

Prestress loss due to elastic shortening [C5.9.5.2.3a-1]

Concrete Creep:

$\gamma_h := 1.7 - RH = 0.95$ — *Factor for relative humidity [5.9.5.3-2]*

$$\gamma_{st} := \frac{5ksi}{1ksi + f'_{ci}} = 1$$

Factor for strength at transfer [5.9.5.3-3]

$$\Delta f_{pCR} := \frac{10 \cdot f_{pj} \cdot \left(N_{ps} \cdot A_{ps1}\right)}{P \cdot W} \cdot \gamma_h \cdot \gamma_{st} = 7.79\,ksi$$

Prestress loss due to concrete creep [first half of 5.9.5.3-1]

Concrete Shrinkage:

$\Delta f_{pSR} := 12ksi \cdot \gamma_h \cdot \gamma_{st} = 11.4\,ksi$ — *Prestress loss due to shrinkage [second half of 5.9.5.3-1]*

Steel Relaxation:

$\Delta f_{pR} := 2.4 \cdot ksi$ — *Prestress loss due to steel relaxation [5.9.5.3]*

Totals:

$\Delta f_{pT} := \Delta f_{pES} + \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pR} = 27.31\,ksi$ — *Total prestress loss*

$f_{pi} := f_{pj} - \Delta f_{pES} = 196.78\,ksi$ — *Effective prestressing steel stress at release*

$f_{pe} := f_{pj} - \Delta f_{pT} = 175.19\,ksi$ — *Effective prestressing steel stress, final*

Check Stresses at Release:

Assume Panel is supported/lifted from casting bed similar to support conditions in the field.

$\sigma_{comp.all} := 0.65f'_{ci} = 2.6\,ksi$

Allowable Compressive stress prior to long-term losses (AASHTO 5.9.4.1)

$\sigma_{tens.all} := 0ksi$

Allowable Tensile Stress prior to long-term losses (Conservative)

$$\sigma_{ptop} := f_{pi} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1}}{S_x} = 0.91\,ksi$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pi} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) - \frac{M_{DC1}}{S_x} = 0.68\,ksi$$

Stress in Bottom Fiber of Panel (Positive is Compression)

$$Check_{top_stress_release} := if\left(\sigma_{ptop} < \sigma_{comp.all} \wedge \sigma_{ptop} > \sigma_{tens.all}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{top_stress_release} = \text{"Top Fiber Stresses OK"}}$$

$$Check_{bot_stress_release} := if\left(\sigma_{pbot} < \sigma_{comp.all} \wedge \sigma_{pbot} > \sigma_{tens.all}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No G}$$

$$\boxed{Check_{bot_stress_release} = \text{"Bottom Fiber Stresses OK"}}$$

Check Stresses at Final:

- ***Full Dead Load plus Full Prestressing***

Assume all losses occur prior to becoming composite

$\sigma_{comp.all_1} := 0.45f'_c = 2.25\,ksi$

Allowable Compressive stress - Dead Load plus Prestressing (AASHTO 5.9.4.2.1)

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1} + M_{DC2} + M_{DC.int}}{S_x} = 1\,ksi$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) - \frac{M_{DC1} + M_{DC2} + M_{DC.int}}{S_x} = 0.42\,ksi$$

Stress in Bottom Fiber of Panel (Positive is Compression)

$$Check_{top_stress_deckpour} := if\left(\sigma_{ptop} < \sigma_{comp.all_1}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{top_stress_deckpour} = \text{"Top Fiber Stresses OK"}}$$

$$Check_{bot_stress_deckpour} := if\left(\sigma_{pbot} < \sigma_{comp.all_1}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{bot_stress_deckpour} = \text{"Bottom Fiber Stresses OK"}}$$

- ***Full Dead Load plus Full Prestressing plus Full Construction Live Load***

$\sigma_{comp.all_2} := 0.60f'_c = 3\,ksi$

Allowable Compressive stress - Dead Load plus Prestressing (AASHTO 5.9.4.2.1)

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1} + M_{DC2} + M_{DC.int} + M_{LL}}{S_x} = 1.13\,ksi$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) - \frac{M_{DC1} + M_{DC2} + M_{DC.int} + M_{LL}}{S_x} = 0.29\,ksi$$

Stress in Bottom Fiber of Panel (Positive is Compression)

$Check_{top_stress_construc} := if\left(\sigma_{ptop} < \sigma_{comp.all_2}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$

$$\boxed{Check_{top_stress_construc} = \text{"Top Fiber Stresses OK"}}$$

$Check_{bot_stress_construc} := if\left(\sigma_{pbot} < \sigma_{comp.all_2}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$

$$\boxed{Check_{bot_stress_construc} = \text{"Bottom Fiber Stresses OK"}}$$

- ***Full Dead Load plus Full Prestressing plus 80% Construction Live Load (Service 3)***

$\sigma_{tens.all_1} := 0ksi$

Allowable Tension stress - Conservative

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1} + M_{DC2} + M_{DC.int} + 0.80M_{LL}}{S_x} = 1.1\,ksi$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) - \frac{M_{DC1} + M_{DC2} + M_{DC.int} + 0.80M_{LL}}{S_x} = 0.31\,ksi$$

Stress in Bottom Fiber of Panel (Positive is Compression)

$Check_{top_stress_SRVIII} := if\left(\sigma_{ptop} > \sigma_{tens.all_1}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$

$$\boxed{Check_{top_stress_SRVIII} = \text{"Top Fiber Stresses OK"}}$$

$Check_{bot_stress_SRVIII} := if\left(\sigma_{pbot} > \sigma_{tens.all_1}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$

$$\boxed{Check_{bot_stress_SRVIII} = \text{"Bottom Fiber Stresses OK"}}$$

Check Composite Panel:

- *Composite Properties:*

$$n := \frac{E_c}{E_{cip}} = 1.08$$

Modular Ratio for Precast and CIP concrete

$$y_bar := \frac{P\cdot\frac{P}{2}\cdot n + (T - P)\cdot[P + 0.5\cdot(T - P)]}{P\cdot n + (T - P)} = 4.17\,in$$

Neutral Axis Location from bottom of panel, gross sections

$$I_c := \frac{W\cdot P^3\cdot n}{12} + \frac{W\cdot(T - P)^3}{12} + W\cdot P\cdot n\cdot\left(y_bar - \frac{P}{2}\right)^2 + (T - P)\cdot W\cdot[T - 0.5\cdot(T - P) - y_bar]^2 = 5097\,in^4$$

$$S_{top} := \frac{I_c}{T - y_bar} = 1178\cdot in^3$$

Section Modulus to top of deck

$$S_{bot} := \frac{I_c}{y_bar\cdot n} = 1134\cdot in^3$$

Section Modulus to Bottom of Deck

$$S_{pan} := \frac{I_c}{(y_bar - P)\cdot n} = 7031\cdot in^3$$

Section Modulus to top of Panel

- *Initial Stresses*

$$\sigma_{top} := 0\,ksi$$

Stress at top of Deck (Fresh concrete therefore no stress yet)

$$\sigma_{pan} := f_{pe}\cdot N_{ps}\cdot A_{ps1}\cdot\left(\frac{1}{P\cdot W} - \frac{ecc}{S_x}\right) + \frac{M_{DC1}}{S_x} + \frac{M_{DC2}}{S_x} + \frac{M_{DC.int}}{S_x} = 1\,ksi$$

Stress at top of panel

$$\sigma_{bot} := f_{pe}\cdot N_{ps}\cdot A_{ps1}\cdot\left(\frac{1}{P\cdot W} + \frac{ecc}{S_x}\right) - \frac{M_{DC1}}{S_x} - \frac{M_{DC2}}{S_x} - \frac{M_{DC.int}}{S_x} = 0.42\,ksi$$

Stress at bottom of panel

Additional Loads - HL-93

- *Live Loads*

  Use AASHTO Table A4-1 and use S=7'-6" and interpolate negative moment section between 3" and 6" as design section is at one quarter of the flange width from the centerline of girder.

Positive:

$M_{LL_pos} := 5.44 \frac{kip \cdot ft}{ft}$ *Positive moment for design span*

Negative:

$M_{neg_3in} := 5.43 \frac{kip \cdot ft}{ft}$ *Negative moment with respective Design Section*

$M_{neg_6in} := 4.61 \frac{kip \cdot ft}{ft}$ *Negative moment with respective Design Section*

$$M_{LL_neg} := M_{neg_3in} + \frac{M_{neg_6in} - M_{neg_3in}}{3in} \cdot \left(\frac{b_f}{4} - 3in \right)$$

$$M_{LL_neg} = 4.88 \cdot \frac{kip \cdot ft}{ft}$$

Max Negative LL moment for design with Impact and Multiple Presence Included

Check Negative Moments (CIP portion only):

- *Treat as mildly reinforced only without compression steel*

$M_{neg} := \frac{1.50 M_{DW.fut}}{W} + 1.75 M_{LL_neg} = 8.84 \cdot \frac{kip \cdot ft}{ft}$ *Negative Moment over Girders, Factored for Strength 1 combos (using positive moment for M_{DW}, conservative)*

$D_{b.trans} := 0.625 in$ *Diameter of transverse reinforcing bar*

$A_{s_bar.trans} := \pi \cdot \frac{D_{b.trans}^2}{4} = 0.31 \, in^2$ *Area of Single transverse bar*

$s_{trans} := 6in$ *Spacing of Reinforcement in the C.I.P. deck longitudinal to the bridge*

$D_{b.long} := 0.625 in$ *Diameter of longitudinal reinforcing bar*

$A_{s_bar.long} := \pi \cdot \frac{D_{b.long}^2}{4} = 0.31 \, in^2$ *Area of Single longitudinal bar*

$s_{long} := 18in$ *Spacing of Reinforcement in the C.I.P. deck longitudinal to the bridge*

$A_{neg} := \frac{A_{s_bar.trans}}{s_{trans}} = 0.61 \cdot \frac{in^2}{ft}$ *Amount of Reinforcement*

$cl := 2.5in$ *Clear Distance to top bars*

$d_e := T - cl - 0.5 \cdot D_{b.trans} = 5.69\,in$ *Depth to reinforcement*

$a := \frac{A_{neg} \cdot F_y}{0.85 f'_c} = 0.72\,in$ *Depth of compression block, assuming Whitney Stress Block and that the stress block remains within the precast panel [AASHTO 5.7.3.1.1-4]*

$M_n := A_{neg} \cdot F_y \cdot \left(d_e - \frac{a}{2}\right) = 16.34 \cdot \frac{kip \cdot ft}{ft}$ *Nominal Negative Moment capacity [AASHTO 5.7.3.2.3]*

$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4ksi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f'_c - 4ksi}{ksi}\right), 0.65\right] & \text{otherwise} \end{cases}$ $\beta_1 = 0.8$ *Stress Block to N.A. Parameter [AASHTO 5.7.2.2]*

$c := \frac{a}{\beta_1}$ $c = 0.9\,in$ *Equivelant Stress Block Depth*

$\varepsilon_s := \frac{0.003}{c} \cdot (d_e - c)$ $\varepsilon_s = 0.0159$ *Strain in 'Tension' Steel at Concrete Crushing*

$\phi_b := \begin{cases} 0.90 & \text{if } \varepsilon_s > 0.005 \\ \max\left[0.75, 0.90 - (0.005 - \varepsilon_s) \cdot \frac{0.90 - 0.75}{0.005 - 0.002}\right] & \text{otherwise} \end{cases}$ $\phi_b = 0.9$ *Resistance Factor for Concrete in Bending [AASHTO 5.5.4.2]*

$\phi_b \cdot M_n = 14.71 \cdot \frac{kip \cdot ft}{ft}$ *Reduced nominal Moment Capacity [AASHTO 5.7.3.2.1)*

$\gamma_1 := 1.6$ $\gamma_3 := 0.67$ *Cracking Moment Coefficients [5.7.3.3.2]*

$M_{cr} := \frac{\gamma_1 \cdot \gamma_3 \cdot f_r \cdot T^2}{6} = 7 \cdot \frac{kip \cdot ft}{ft}$ *Cracking Moment [5.7.3.3.2]*

$Check_{Deck_STR} := if\left(\phi_b \cdot M_n \ge M_{neg}, \text{"Deck is OK for Strength"}, \text{"Deck is No Good for Strength"}\right)$

$Check_{Deck_STR} = \text{"Deck is OK for Strength"}$

$Check_{Deck_min_reinf} := if\left[\left(\phi_b \cdot M_n \ge M_{cr}\right) \vee \left(\phi_b \cdot M_n \ge 1.33 M_{neg}\right), \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No (}\right.$

$Check_{Deck_min_reinf} = \text{"Min. Reinforcement is OK"}$ *[AASHTO 5.7.3.3.2]*

Crack Control Check (Negative Moment):

$M_{s.neg} := \frac{M_{DW.fut}}{W} + M_{LL_neg} = 5.08 \cdot \frac{kip \cdot ft}{ft}$ *Service Negative moment*

$\rho := \frac{A_{neg}}{d_e} = 0.009$ *Reinforcement Ratio, ignoring compression steel*

$n := \frac{E_s}{E_{cip}} = 7.31$ *Modular Ratio*

$k := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n = 0.3$ *Ratio of depth to neutral axis over depth to tension reinforcement*

$j := 1 - \frac{k}{3} = 0.9$ *Ratio of distance between compression and tensior centroids to depth to tension steel*

$f_{ss} := \frac{M_{s.neg}}{A_{neg} \cdot j \cdot d_e} = 19.42 \cdot ksi$ *Stress in tension reinforcement at service moment*

$Check_{fss} := if\left(f_{ss} < 0.6 \cdot F_y, \text{"Reinf. Ok"}, \text{"Reinf. No Good"}\right)$ $\boxed{Check_{fss} = \text{"Reinf. Ok"}}$

$d_c := T - d_e + T_{int}$ $d_c = 3.31\,in$ *Distance to CG Steel*

$\beta_s := 1 + \frac{d_c}{0.70 \cdot \left(T + T_{int} - d_c\right)}$ $\beta_s = 1.83$ *Crack Parameter [AASHTO 5.7.3.4]*

$\gamma_e := 1.0$ *Class 1 Exposure = 1.0*
Class 2 Exposure = 0.75

$s := \frac{700 \cdot \frac{kip}{in} \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ $s = 13.05\,in$ $s_{trans} = 6\,in$

$Check_{Deck_crack} := if\left(s_{trans} < s, \text{"Reinforcement Spacing is OK"}, \text{"Reinforcement Spacing is No Good"}\right)$

$\boxed{Check_{Deck_crack} = \text{"Reinforcement Spacing is OK"}}$

Check Positive Moments:

- *Assume compression block is entirely within C.I.P. portion*

$$M_{u.pos} := \frac{1.25}{W}\cdot\left(M_{DC1} + M_{DC2} + M_{DC.int}\right) + \frac{1.50}{W}\cdot\left(M_{DW.fut}\right) + 1.75M_{LL_pos} = 10.55\cdot\frac{kip\cdot ft}{ft}$$

Positive Moment between Girders, Factored for Strength 1 combos

$$A'_s := A_{neg} = 0.61\cdot\frac{in^2}{ft}$$

Compression Steel per Foot

$$A_{ps} := \frac{A_{ps1}\cdot N_{ps}}{W} = 0.17\cdot\frac{in^2}{ft}$$

Prestressing steel per foot

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_{cip} \le 4ksi \\ \max\left[0.85 - 0.05\cdot\left(\frac{f'_{cip} - 4ksi}{ksi}\right), 0.65\right] & \text{otherwise} \end{cases} \qquad \beta_1 = 0.85$$

Stress Block to N.A. Parameter [AASHTO 5.7.2.2]

$$k := 2\cdot\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$

Prestress parameter [AASHTO 5.7.3.1.1-2]

$$d_p := T - \frac{P}{2} = 6.75\,in$$

Depth to Prestressing Steel from extreme compresion fiber

$$d'_s := cl + D_{b.trans}\cdot 0.5 = 2.81\,in$$

Depth to Compression steel from extreme compression fiber

Guess

$$c_g := 1.5in$$

Given

$$c_g = \frac{A_{ps}\cdot f_{pu} - A'_s\cdot\max\left[\min\left[\left(\frac{c_g - d'_s}{c_g}\right)\cdot 0.003\cdot E_s, F_y\right], -F_y\right]}{0.85\cdot f'_{cip}\cdot\beta_1 + k\cdot A_{ps}\cdot\frac{f_{pu}}{d_p}}$$

[AASHTO 5.7.3.1.1-4]

$$c := \text{Find}(c_g) = 1.93\,in$$

Depth to Neutral Axis from Extreme Compression Fiber

$$f'_s := \max\left[\min\left[\left(\frac{c - d'_s}{c}\right)\cdot 0.003\cdot E_s, F_y\right], -F_y\right] = -40.04\,ksi$$

Stress in Compression Steel (negative value indicates tension forces are present)

$$a := \beta_1\cdot c = 1.64\,in$$

Compression Block Depth (5.7.3.2.2)

$$f_{ps} := f_{pu}\cdot\left(1 - k\cdot\frac{c}{d_p}\right) = 248.43\,ksi$$

Stress in tendons at ultimate moment [AASHTO 5.7.3.1.1-1]

$$M_{n.pos} := A_{ps}\cdot f_{ps}\cdot\left(d_p - \frac{a}{2}\right) - A'_s\cdot f'_s\cdot\left(d'_s - \frac{a}{2}\right) = 24.96\cdot\frac{kip\cdot ft}{ft}$$

Positive Moment Capacity of Deck [AASHTO 5.7.3.2.2-1]

$$f_{cpe} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) = 0.71\,ksi$$

Compressive stress at extreme fiber where external loads cause tension [5.7.3.3.2]

$$M_{dnc} := M_{DC1} + M_{DC2} + M_{DC.int} = 4.75 \cdot kip \cdot ft$$

$\gamma_1 := 1.6$ $\gamma_2 := 1.1$ $\gamma_3 := 1.0$

Cracking Moment Coefficients [5.7.3.3.2]

$$M_{cr.pos} := \left[\gamma_3 \cdot S_{bot} \cdot \left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe} \right) - M_{dnc} \cdot \left(\frac{S_{bot}}{S_x} - 1 \right) \right] \cdot \frac{1}{W} = 16.5 \cdot \frac{kip \cdot ft}{ft}$$

Cracking Moment of Panel (5.7.3.3.2-1)

$$\phi_b := \min\left[0.65 + 0.15 \cdot \left(\frac{d_p}{c} - 1 \right), 0.90 \right] = 0.9$$

Resistance factor for bending, assuming member to be non-prestressed (conservative), (5.5.4.2.1-2)

$$Check_{Panel_STR} := if\left(\phi_b \cdot M_{n.pos} \ge M_{u.pos}, \text{"Deck is OK for Strength"}, \text{"Deck is No Good for Strength"} \right)$$

$$\boxed{Check_{Panel_STR} = \text{"Deck is OK for Strength"}}$$

$$Check_{Panel_min_reinf} := if\left[\left(\phi_b \cdot M_{n.pos} \ge M_{cr.pos} \right) \vee \left(\phi_b \cdot M_{n.pos} \ge 1.33 M_{u.pos} \right), \text{"Min. Reinforcement is OK"}, \text{"Min. Rein.} \right.$$

$$\boxed{Check_{Panel_min_reinf} = \text{"Min. Reinforcement is OK"}}$$

[AASHTO 5.7.3.3.2]

Temperature And Shrinkage Steel:

$$A_{ts.min} := \min\left[\max\left[0.11 \cdot in^2, \left[\frac{1.30 \cdot 35ft \cdot T}{2 \cdot (35ft + T) \cdot F_y} \right] \cdot \frac{kip}{in} \right], 0.60in^2 \right]$$

Temperature and Shrinkage Steel Boundaries [AASHTO 5.10.8]

$$A_{ts.min} = 0.11 \cdot in^2$$

$$A_{s.top} := \min\left(\frac{A_{s_bar.trans}}{s_{trans}}, \frac{A_{s_bar.long}}{s_{long}} \right) \qquad A_{s.top} = 0.2 \cdot \frac{in^2}{ft}$$

Reinforcement Provided in Top of Deck, Parallel with Traffic (#5's at 18")

$$Check_{TS_Steel} := if\left(A_{s.top} > \frac{A_{ts.min}}{W}, \text{"Temperature and Shrinkage Steel OK"}, \text{"Temperature and Shrinkage Steel No Good"} \right)$$

$$\boxed{Check_{TS_Steel} = \text{"Temperature and Shrinkage Steel OK"}}$$

$$S_{ts.max} := \min(18in, 3 \cdot T) \qquad S_{ts.max} = 18 \cdot in$$

Maximum Spacing of T&S Reinforcement

$$Check_{Reinf_Spa} := if\left(\max\left(s_{trans}, s_{long} \right) \le S_{ts.max}, \text{"Reinforcement Spacing OK for T\&S"}, \text{"Reinforcement Spacing No Good for} \right.$$

$$\boxed{Check_{Reinf_Spa} = \text{"Reinforcement Spacing OK for T\&S"}}$$

Distribution Steel:

$$A_{dist} := \frac{A_{bar}}{S_{bar}} = 0.22 \cdot \frac{in^2}{ft}$$

Amount of distribution steel provided in the precast deck panels

$$A_{dist.min} := \min\left(\frac{220}{\sqrt{\frac{S}{ft}}}, 0.67\right) \cdot \frac{A_{ps1} \cdot N_{ps}}{W} = 0.11 \cdot \frac{in^2}{ft}$$

Minimum amount of distribution steel [AASHTO 9.7.3.2]

$$Check_{Dist_Steel} := \text{if}\left(A_{dist} \geq A_{dist.min}, \text{"Distribution Steel OK"}, \text{"Distribution Steel No Good"}\right)$$

$$\boxed{Check_{Dist_Steel} = \text{"Distribution Steel OK"}}$$

Stress Checks

- ***Full Dead Load plus Full Prestressing***

$$\sigma_{comp.all_p} := 0.45 f'_c = 2.25\,ksi$$

Allowable Compressive stress - Dead Load plus Prestressing (AASHTO 5.9.4.2.1)

$$\sigma_{comp.all_cip} := 0.45 f'_{cip} = 1.8\,ksi$$

Allowable Compressive stress - Dead Load plus Prestressing in CIP portion (AASHTO 5.9.4.2.1)

$$\sigma_{top} := \frac{M_{DW.fut}}{S_{top}} = 0.02\,ksi$$

Stress in Top Fiber of Deck (Positive is Compression)

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x}\right) + \frac{M_{DC1}}{S_x} + \frac{M_{DC2}}{S_x} + \frac{M_{DC.int}}{S_x} - \frac{M_{DW.fut}}{S_{pan}} = 1\,ksi$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x}\right) + \frac{M_{DW.fut}}{S_{bot}} = 0.73\,ksi$$

Stress in Bottom Fiber of Panel (Positive is Compression, near girder)

$$Check_{Deck_Dead_Stress} := \text{if}\left(\sigma_{top} < \sigma_{comp.all_cip}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Deck_Dead_Stress} = \text{"Top Fiber Stresses OK"}}$$

$$Check_{Panel_Dead_TopStress} := \text{if}\left(\sigma_{ptop} < \sigma_{comp.all_p}, \text{"Panel Top Fiber Stresses OK"}, \text{"Panel Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_Dead_TopStress} = \text{"Panel Top Fiber Stresses OK"}}$$

$$Check_{Panel_Dead_BotStress} := \text{if}\left(\sigma_{pbot} < \sigma_{comp.all_p}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_Dead_BotStress} = \text{"Bottom Fiber Stresses OK"}}$$

- ***Full Dead Load plus Full Prestressing plus Full Live Load***

$\sigma_{comp.all_p} := 0.60f'_c = 3\,ksi$

Allowable Compressive stress - Dead Load plus Prestressing (AASHTO 5.9.4.2.1)

$\sigma_{comp.all_p} := 0.60f'_{cip} = 2.4\,ksi$

Allowable Compressive stress in CIP portion - Dead Load plus Prestressing (AASHTO 5.9.4.2.1)

$$\sigma_{top} := \frac{M_{DW.fut}}{S_{top}} + \frac{M_{LL_pos} \cdot W}{S_{top}} = 0.46\,ksi$$

Stress in Top Fiber of Deck (Positive is Compression)

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1}}{S_x} + \frac{M_{DC2}}{S_x} + \frac{M_{DC.int}}{S_x} - \frac{M_{DW.fut}}{S_{pan}} - \frac{M_{LL_pos} \cdot W}{S_{pan}} = 0.92\,ksi$$

Stress in Top Fiber of Panel near midspan (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) + \frac{M_{DW.fut}}{S_{bot}} + \frac{M_{LL_neg} \cdot W}{S_{bot}} = 1.14\,ksi$$

Stress in Bottom Fiber of Panel near girder (Positive is Compression)

$$Check_{Deck_DandL_Stress} := \text{if}\left(\sigma_{top} < \sigma_{comp.all_cip}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Deck_DandL_Stress} = \text{"Top Fiber Stresses OK"}}$$

$$Check_{Panel_DandL_TopStress} := \text{if}\left(\sigma_{ptop} < \sigma_{comp.all_p}, \text{"Panel Top Fiber Stresses OK"}, \text{"Panel Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_DandL_TopStress} = \text{"Panel Top Fiber Stresses OK"}}$$

$$Check_{Panel_DandL_BotStress} := \text{if}\left(\sigma_{pbot} < \sigma_{comp.all_p}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_DandL_BotStress} = \text{"Bottom Fiber Stresses OK"}}$$

- ***Full Dead Load plus Full Prestressing plus 80% Live Load (Service 3)***

$\sigma_{tens.all_p} := 0$ *Allowable Tension stress in panel - Conservative*

The top fiber stresses of the Cast in place portion have already been investigated through spacing limitations of the mild reinforcement outlined in AASHTO 5.7.3.4.

$$\sigma_{ptop} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} - \frac{ecc}{S_x} \right) + \frac{M_{DC1}}{S_x} + \frac{M_{DC2}}{S_x} + \frac{M_{DC.int}}{S_x} - \frac{M_{DW.fut}}{S_{pan}} - \frac{0.80 M_{LL}}{S_{pan}} = 0.99\,\text{ksi}$$

Stress in Top Fiber of Panel (Positive is Compression)

$$\sigma_{pbot} := f_{pe} \cdot N_{ps} \cdot A_{ps1} \cdot \left(\frac{1}{P \cdot W} + \frac{ecc}{S_x} \right) - \frac{M_{DC1}}{S_x} - \frac{M_{DC2}}{S_x} - \frac{M_{DC.int}}{S_x} - \frac{M_{DW.fut}}{S_{bot}} - \frac{0.80 M_{LL_pos} \cdot W}{S_{bot}} = 0.03\,\text{ksi}$$

Stress in Bottom Fiber of Panel (Positive is Compression)

$$Check_{Panel_SRVIII_TopStress} := \text{if}\left(\sigma_{ptop} > \sigma_{tens.all_p}, \text{"Top Fiber Stresses OK"}, \text{"Top Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_SRVIII_TopStress} = \text{"Top Fiber Stresses OK"}}$$

$$Check_{Panel_SRVIII_BotStress} := \text{if}\left(\sigma_{pbot} > \sigma_{tens.all_p}, \text{"Bottom Fiber Stresses OK"}, \text{"Bottom Fiber Stresses No Good"}\right)$$

$$\boxed{Check_{Panel_SRVIII_BotStress} = \text{"Bottom Fiber Stresses OK"}}$$

T·Y·LIN INTERNATIONAL
engineers | planners | scientists

PROJECT: Waitsfield **JOB NO:** 411847
TASK: Deck Overhang Design
DESIGN: T. Poulin **DATE:** Dec. 2015
CHECK: J. Olund **DATE:** Dec. 2015

Deck Overhang Design

References:

- *Ref: AASHTO A13.4*
- *Rail loading is for TL4*
- *Barrier design is standard NETC 2 Rail*
- *Design as Post-and-Beam Railing*

Assumptions and Methodology

Assumptions:

- *Two Critical Locations to Investigate: 1) Gutterline (2'-0" from Edge of Deck); 2) Face of Web*
- *Use minimum overhang for the section under investigation (conservative for bar development)*
- *45deg load distribution through deck, from back of post base plate to face of web*
- *All railing loads are transmitted to the nearest post*

Methodology:

Three Cases to Investigate:

- *Case 1: Extreme Event II w/Transverse & Longitudinal Forces from A13.2 for Specified Rail Test rating from Above*
- *Case 2: Extreme Event II with Vertical Forces from A13.2 for Specified Rail Test Rating from Above*
- *Case 3: Strength I Limit State with Dead and Live Loads*

Material Properties:

$f'_c := 4.0\text{ksi}$ — *Strength of Concrete (HPC A Concrete)*

$f_y := 60\text{ksi}$ — *Yield Strength of Rebar*

$E_s := 29000\text{ksi}$ — *Steel Modulus of Elasticity*

$E_c := 2500\text{ksi}\cdot\left(\frac{f'_c}{\text{ksi}}\right)^{\frac{1}{3}}$ $\quad E_c = 3969\cdot\text{ksi}$ — *Concrete Modulus of Elasticity [AASHTO C5.4.2.4-1]*

$f_r := 0.24\sqrt{f'_c\cdot\text{ksi}}$ $\quad f_r = 480\cdot\text{psi}$ — *Rupture Stress of Concrete [AASHTO 5.4.2.6]*

$D_{bar.top_prov} := 0.625\text{in}$ — *Top Reinforcing Bar Diameter - provided from deck reinforcement*

$A_{bar.top_prov} := \pi D_{bar.top_prov}^2 \div 4 = 0.31\cdot\text{in}^2$ — *Top Reinforcing Bar Area - provided from deck reinforcement*

$D_{bar.bot_prov} := 0.625\text{in}$ — *Bottom Reinforcing Bar Diameter - provided from deck reinforcement*

$A_{bar.bot_prov} := \pi D_{bar.bot_prov}^2 \div 4 = 0.307\cdot\text{in}^2$ — *Bottom Reinforcing Bar Area - provided from deck reinforcement*

$S_{bar.top_prov} := 6.0\text{in}$ — *Bar Spacing on Top of Slab - provided from deck reinforcement*

$S_{bar.bot_prov} := 6.0\text{in}$ — *Bar Spacing on Bottom of Slab - - provided from deck reinforcement*

$D_{bar.top_add} := 0in$ — *Top Reinforcing Bar Diameter - provided from additional overhang reinforcement*

$A_{bar.top_add} := \pi D_{bar.top_add}^2 \div 4 = 0 \cdot in^2$ — *Top Reinforcing Bar Area - provided from additional overhang reinforcement*

$D_{bar.bot_add} := 0.625in$ — *Bottom Reinforcing Bar Diameter - provided from additional overhang reinforcement*

$A_{bar.bot_add} := \pi D_{bar.bot_add}^2 \div 4 = 0.307 \cdot in^2$ — *Bottom Reinforcing Bar Area - provided from additional overhang reinforcement*

$S_{bar.top_add} := 99999in$ *effectively no bars* — *Bar Spacing on Top of Slab - provided from additional overhang reinforcement*

$S_{bar.bot_add} := 6.0in$ — *Bar Spacing on Bottom of Slab - - provided from additional overhang reinforcement*

Geometry:

Bridge

$t_{slab} := 8.5in$ — *Structural Thickness of Deck*

$L_{over} := 2ft + 6in$ — *Length of Overhang from Center of Web*

$t_{asph} := 3in$ — *Future Wearing Surface Thickness (VTrans SDM 3.3.2)*

$b_f := 20in$ — *Width of Top Flange*

$S_{gird} := 7ft + 6in$ — *Girder Spacing*

$t_w := 0.75in$ — *Web Thickness*

$edge := 7.625in - 1.8125in = 5.812 \cdot in$ $\quad edge = 5.812 \cdot in$ — *Distance beyond Outer Edge of Post Base Plate*

$edge_{post} := 7.625in$ — *Distance beyond post flange*

$W_{curb} := 2.0ft$ — *Width of Curb*

$t_{curb} := 7.0in + 0.021 \cdot 0.5 \cdot W_{curb} = 7.252 \cdot in$ — *Average Thickness of Curb*

$d := t_{slab} - 2.5in - 0.5 \cdot \max(D_{bar.top_prov}, D_{bar.top_add}) = 5.688 \cdot in$ — *Distance from extreme compression fiber to center of tensile reinforcement*

$d' := 1.5in + 0.5 \cdot \max(D_{bar.bot_prov}, D_{bar.bot_add}) = 1.812 \cdot in$ — *Distance from extreme compression fiber to center of compression reinforcement*

Rail

$b_{rail} := 10.375in = 10.375 \cdot in$ — *Barrier Width*

$h_{rail} := 2ft + 3in$ — *Barrier Rail Height*

$S_{post} := 7ft + 4.5in$ — *Bridge Rail Post Spacing (maximum)*

$F_v := 50ksi$ — *Yield strength of post*

$A_f := 6.08in \cdot 0.455in = 2.766 \cdot in^2$ — *Area of post flange*

$M_{post} := F_y \cdot 18.9in^3 = 78.75 \cdot kip \cdot ft$ — *Post Plastic Moment (W6 x 25)*

$V_{post} := 0.58 \cdot F_y \cdot 6in \cdot 0.32in = 55.68 \cdot kip$ — *Post Plastic Shear Strength (W6 x 25)*

$W_b := 14in$ — *Width of Post Base Plate*

$L_b := 10in$ — *Length of Post Base Plate*

$t_b := 1in$ — *Thickness of Post Base Plate*

$d_b := \left(1 + \frac{13}{16} + 6 + \frac{3}{8}\right)in = 8.188 \cdot in$ — *Distance from Outer Edge of Base Plate to Inner Anchor Bolts*

$x_g := \left(W_{curb} - 2.3125in\right) = 21.688 \cdot in$ — *Distance from Outer Edge curb to curb stirrups at gutterline*

$Rail_h := \begin{pmatrix} 11 \\ 23 \end{pmatrix} in$ $\quad Rail_M := \begin{pmatrix} 4.69 \\ 9.91 \end{pmatrix} in^3 \cdot 46ksi$ — *Rail Heights and Plastic Moments (HSS4x4x0.25 and HSS8x4x0.3125)*

$$y_{bar} := \frac{\sum \overrightarrow{\left(Rail_h \cdot Rail_M\right)}}{\sum Rail_M} = 19.145 \cdot in$$

Centroid of Rail Resistance

Load Distribution Dimensions

$x_g = 21.688 \cdot in$ — *Distance from Outer Edge of Base Plate to Gutterline*

$x_f := L_{over} - 2ft + x_g$ $\quad x_f = 2.307\ ft$ — *Distance from Face of Web to curb stirrups*

$L_{g_1} := \min\left[W_b + 2 \cdot t_{curb} + \left(2x_g\right), S_{post}\right]$ $\quad L_{g_1} = 5.99\ ft$ — *Length of Deck Resisting Forces at Gutterline (Load Case 1)*

$L_{g_2} := \min\left(W_b + 2 \cdot t_{curb} + 4 \cdot x_g, S_{post}\right)$ $\quad L_{g_2} = 7.375\ ft$ — *Length of Deck Resisting Forces at Gutterline (Load Case 2)*

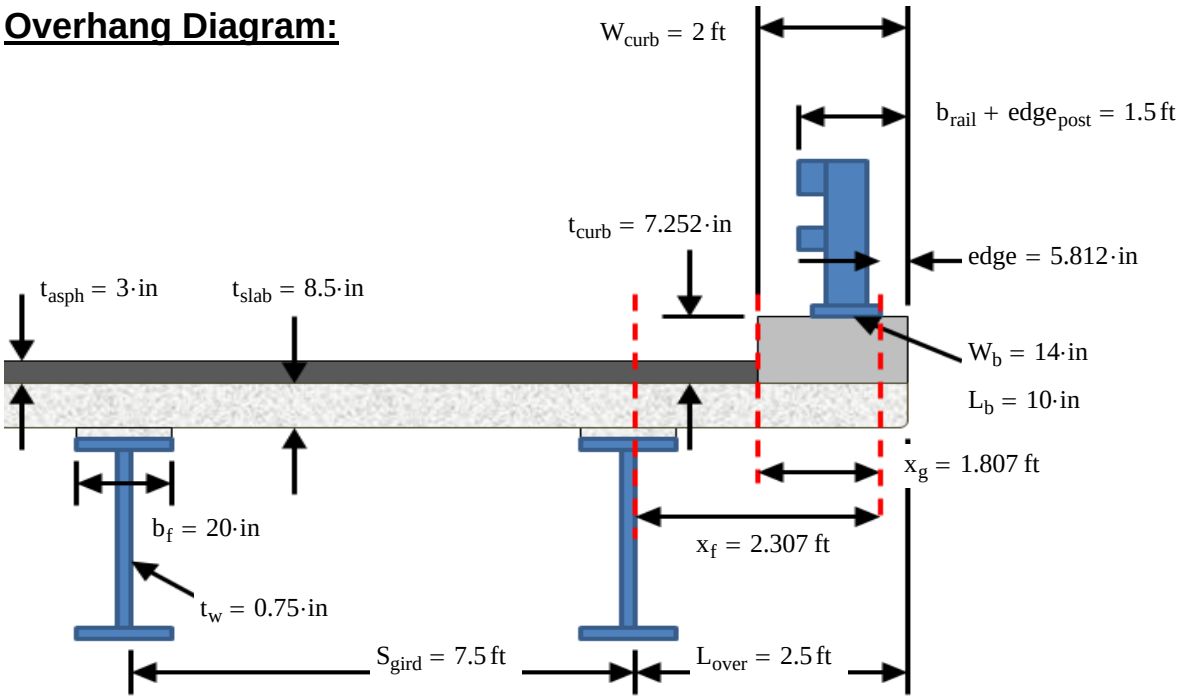
$L_{f_1} := \min\left(L_{g_1} + 2 \cdot x_f, S_{post}\right)$ $\quad L_{f_1} = 7.375\ ft$ — *Length of Deck Resisting Forces at Face of Web (Load Case 1)*

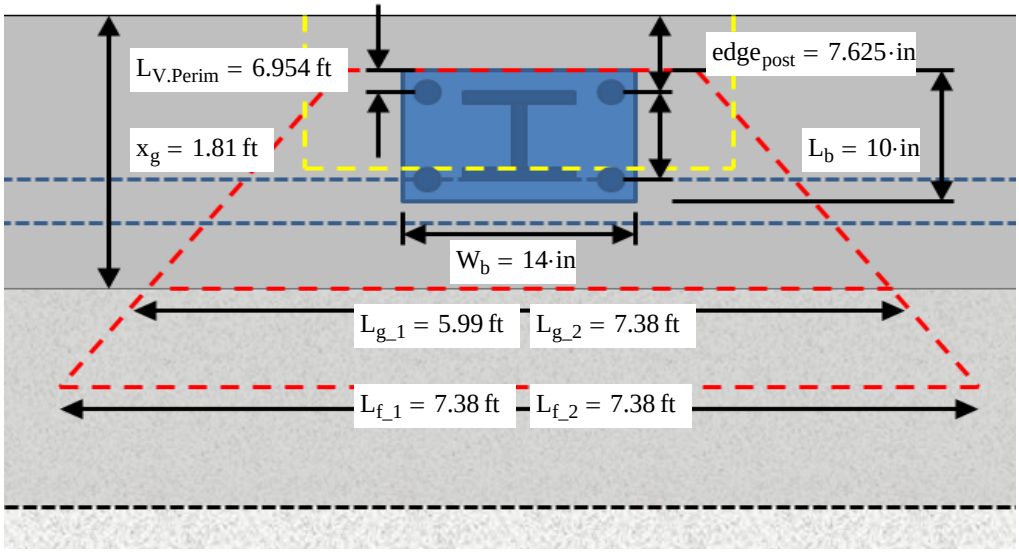
$L_{f_2} := \min\left(L_{g_2} + 2 \cdot x_f, S_{post}\right)$ $\quad L_{f_2} = 7.375\ ft$ — *Length of Deck Resisting Forces at Face of Web (Load Case 2)*

$$L_{V.Perim} := \left(W_b + 2 \cdot t_{curb} + t_{slab}\right) + 2 \cdot \left[\left(\frac{d_b}{2} + edge_{post}\right) + t_{curb} + \frac{t_{slab}}{2}\right] = 6.954\ ft$$

Length of Punching Shear Perimeter, for Load Case 1 (AASHTO A13.4.3.2)

Overhang Diagram:





Loads:

$\gamma_{conc} := 150pcf$ *Unit Weight of Concrete Deck and Curb*

$\gamma_{asph} := 140pcf$ *Unit Weight of Wearing Surface*

$$w_{rail} := 1.05\left(\frac{25plf \cdot h_{rail} + L_b \cdot W_b \cdot t_b \cdot 495pcf}{S_{post}} + 12.18plf + 23.32plf\right)$$

$w_{rail} = 50.993 \cdot plf$ *Weight of Railing (without curb)*

$F_t := 54kip$ *Transverse Load for Test Level [AASHTO Table A13.2-1]*

$F_L := 18kip$ *Longitudinal Load for Rail Test Level [AASHTO Table A13.2-1]*

$F_v := 18kip$ *Vertical Load for Rail Test Level [AASHTO Table A13.2-1]*

$L_t := 3.5ft$ *Transverse Load Distribution Length*

$L_L := 3.5ft$ *Longitudinal Load Distribution Length*

$L_v := 18ft$ *Vertical Load Distribution Length*

$M_{post} = 78.75 \cdot kip \cdot ft$ *Plastic Moment Resistance of Single Post*

$P_{post} := \frac{M_{post}}{y_{bar}}$ $P_{post} = 49.36 \cdot kip$ *Shear Force on Single Post Corresponding to M_{post}*

Load Factors:

$IM := 1.33$ *Impact Factor (Impact Case 3 Only)*

$\gamma_{DC.str} := 1.25$ *Strength I Load Factor, DC*

$\gamma_{DW.str} := 1.50$ *Strength I Load Factor, DW*

$\gamma_{LL.str} := 1.75$ *Strength I Load Factor, LL*

$\gamma_{DC.ex} := 1.25$ *Extreme II Load Factor, DC [AASHTO A13.4.1]*

$\gamma_{DW.ex} := 1.50$ *Extreme II Load Factor, DW [AASHTO A13.4.1]*

$\gamma_{LL.ex} := 0.50$ *Extreme II Load Factor, LL [AASHTO 3.4.1]*

$\gamma_{CT.ex} := 1.0$ *Extreme II Load Factor, CT (AASHTO 3.4.1)*

$MP := 1.20$ *Multiple Presence Factor (AASHTO 3.6)*

Gutterline Location Moments (unfactored):

DC

$$M_{DC.g} := w_{rail}\cdot\left(x_g - \frac{L_b}{2}\right) + \gamma_{conc}\cdot\left(t_{slab} + t_{curb} + .5in\right)\cdot\left(W_{curb}\right)\cdot\frac{\left(W_{curb}\right)}{2} = 5.727\cdot\frac{kip\cdot in}{ft}$$

Moment Due to Self Weight

DW

$$M_{DW.g} := 0$$

Moment Due to wearing surface

LL Case 1

$$T_{CT1.g} := \frac{P_{post}}{L_{g_1}} \qquad T_{CT1.g} = 8.24\cdot\frac{kip}{ft}$$

Thrust applied due to Case 1

$$M_{CT1.g} := \frac{M_{post}}{L_{g_1}} \qquad M_{CT1.g} = 157.765\cdot\frac{kip\cdot in}{ft}$$

Applied Moment per Unit Foot due to Case 1 Loading

LL Case 2

$$P_{v_2} := \frac{F_v\cdot S_{post}}{L_v} \qquad P_{v_2} = 7.375\cdot kip$$

Vertical Force per Post for Punching Shear Case 2

$$M_{CT2.g} := \frac{P_{v_2}\cdot x_g}{L_{g_2}} \qquad M_{CT2.g} = 1.8\cdot\frac{kip\cdot ft}{ft}$$

Moment per Unit Width due to Case 2 Loading

$$P_{CT2.g} := P_{v_2} \qquad P_{CT2.g} = 7.375\cdot kip$$

Punching Shear

LL Case 3

Live Load application does not affect moments at Gutterline Location.

Design Forces at Gutterline:

$$M_{ex_g} := \begin{pmatrix} \gamma_{DC.ex} \cdot M_{DC.g} + \gamma_{DW.ex} \cdot M_{DW.g} + M_{CT1.g} \\ \gamma_{DC.ex} \cdot M_{DC.g} + \gamma_{DW.ex} \cdot M_{DW.g} + M_{CT2.g} \end{pmatrix} = \begin{pmatrix} 13.744 \\ 2.404 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft}$$

Factored Moment per Unit Width under Extreme Event 2

$P_{ex_g} := \gamma_{CT.ex} \cdot P_{CT2.g}$ $\quad P_{ex_g} = 7.375 \cdot kip$ *Maximum Factored Shear Force for Punching Shear Design*

$T_{ex_g} := \gamma_{CT.ex} \cdot T_{CT1.g}$ $\quad T_{ex_g} = 8.24 \cdot \frac{kip}{ft}$ *Maximum Factored Tension Force per Unit Width*

$M_{u_g} := \gamma_{DC.str} \cdot M_{DC.g} + \gamma_{DW.str} \cdot M_{DW.g}$ $\quad M_{u_g} = 0.597 \cdot \frac{kip \cdot ft}{ft}$ *Moment at Gutterline under Strength 1*

$M_{s_g} := M_{DC.g} + M_{DW.g}$ $\quad M_{s_g} = 0.477 \cdot \frac{kip \cdot ft}{ft}$ *Moment at Gutterline under Service 1*

Moments at Face of Web of Exterior Beam (unfactored):

DC

$$M_{DC.f} := w_{rail} \cdot \left(x_f - \frac{L_b}{2}\right) + \gamma_{conc} \cdot (t_{slab} + t_{curb} + .5in) \cdot (W_{curb}) \cdot \left(L_{over} - \frac{t_w}{2} - \frac{W_{curb}}{2}\right) \ldots = 8.466 \cdot \frac{kip \cdot in}{ft}$$
$$+ \gamma_{conc} \cdot (t_{slab} + .5in) \cdot \frac{\left(L_{over} - \frac{t_w}{2} - W_{curb}\right)^2}{2}$$

Moment Due to Self Weight

DW

$$M_{DW.f} := \gamma_{asph} \cdot t_{asph} \cdot \frac{\left(L_{over} - W_{curb} - \frac{t_w}{2}\right)^2}{2} = 0.046 \cdot \frac{kip \cdot in}{ft}$$

Moment Due to wearing surface

LL Case 1

$T_{CT1.f} := \frac{P_{post}}{L_{f_1}}$ $\quad T_{CT1.f} = 6.693 \cdot \frac{kip}{ft}$ *Thrust applied due to Case 1*

$M_{CT1.f} := \frac{M_{post}}{L_{f_1}}$ $\quad M_{CT1.f} = 10.678 \cdot \frac{kip \cdot ft}{ft}$ *Applied Moment per Unit Foot due to Case 1 Loading*

LL Case 2

$P_v := \frac{F_v \cdot S_{post}}{L_v}$ $\quad P_v = 7.375 \cdot kip$ *Vertical Force per Post for Punching Shear Case 2*

$M_{CT2.f} := \frac{P_v \cdot x_f}{L_{f_2}}$ $\quad M_{CT2.f} = 2.3 \cdot \frac{kip \cdot ft}{ft}$ *Moment per Unit Width due to Case 2 Loading*

$P_{CT2.f} := P_v$ $\quad P_{CT2.f} = 7.375 \cdot kip$ *Punching Shear*

LL Case 3

$D_{LL} := \max\left(L_{over} - W_{curb} - 1ft - \frac{t_w}{2}, 0\right)$ $\quad D_{LL} = 0 \cdot in$ *Distance Between Live Load Application and Point Of Interest*

$W_d := 45in + \frac{10}{12} \cdot D_{LL} = 45 \cdot in$ *Load Distribution for point load (AASHTO Table 4.6.2.1.3-1)*

$$V_{LL3.f} := \max\left[MP \cdot \left[if\left[D_{LL} = 0, 0kip, \max\left[\frac{32kip \cdot IM}{(14ft + W_d)}, \frac{16kip \cdot IM}{(W_d)}, \frac{2 \cdot 12.5kip \cdot IM}{(W_d + 4ft)}\right] + 64psf \cdot (D_{LL} + 1ft)\right]\right], 0kip \cdot \frac{ft}{ft}\right] = 0 \cdot \frac{ki}{f}$$

Shear at face of Web (AASHTO 3.6.1.3.4) - based on entire truck length, single HL93 wheel, or pair of tandem wheels

$$M_{LL3.f} := \max\left[MP \cdot \left[\max\left[D_{LL} \cdot \frac{32kip \cdot IM}{(14ft + W_d)}, D_{LL} \cdot \frac{16kip \cdot IM}{(W_d)}, D_{LL} \cdot \frac{2 \cdot 12.5kip \cdot IM}{(W_d + 4ft)}\right] + 64psf \cdot \frac{(D_{LL} + 1ft)^2}{2}\right], 0kip \cdot \frac{ft}{ft}\right] = 0.04 \cdot \frac{ki}{f}$$

Moment at face of Web (AASHTO 3.6.1.3.4) - based on entire truck length, single HL93 wheel, or pair of tandem wheels

Design Forces at Face of Web:

$$M_{ex_f} := \begin{pmatrix} \gamma_{DC.ex} \cdot M_{DC.f} + \gamma_{DW.ex} \cdot M_{DW.f} + M_{CT1.f} + \gamma_{LL.ex} \cdot M_{LL3.f} \\ \gamma_{DC.ex} \cdot M_{DC.f} + \gamma_{DW.ex} \cdot M_{DW.f} + M_{CT2.f} + \gamma_{LL.ex} \cdot M_{LL3.f} \end{pmatrix} = \begin{pmatrix} 11.585 \\ 3.214 \end{pmatrix} \cdot \frac{kip \cdot ft}{ft}$$

Factored Moment per Unit Width under Extreme Event 2

$P_{ex_f} := \gamma_{CT.ex} \cdot P_{CT2.f}$ $\quad P_{ex_f} = 7.375 \cdot kip$ *Maximum Factored Shear Force for Punching Shear Design*

$T_{ex_f} := \gamma_{CT.ex} \cdot T_{CT1.f}$ $\quad T_{ex_f} = 6.693 \cdot \frac{kip}{ft}$ *Maximum Factored Tension Force per Unit Width*

$M_{u_f} := \gamma_{DC.str} \cdot M_{DC.f} + \gamma_{DW.str} \cdot M_{DW.f} + \gamma_{LL.str} \cdot M_{LL3.f}$

$M_{u_f} = 0.955 \cdot \frac{kip \cdot ft}{ft}$ *Moment at Face of Web under Strength 1*

$M_{s_f} := M_{DC.f} + M_{DW.f} + M_{LL3.f}$ $\quad M_{s_f} = 0.748 \cdot \frac{kip \cdot ft}{ft}$ *Moment at Face of Web under Service 1*

Overhang Design Forces:

$M_s := \max(M_{s_g}, M_{s_f})$ $\quad \boxed{M_s = 0.748 \cdot kip \cdot \frac{ft}{ft}}$ *Service Moment for Crack Control.*

$M_u := \max(M_{u_g}, M_{u_f})$ $\quad \boxed{M_u = 0.955 \cdot \frac{kip \cdot ft}{ft}}$ *Maximum Moment in Overhang under Strength*

$M_{ex} := \max(M_{ex_g_0}, M_{ex_g_1}, M_{ex_f_0}, M_{ex_f_1})$ $\quad \boxed{M_{ex} = 13.744 \cdot \frac{kip \cdot ft}{ft}}$ *Maximum Moment in Overhang under Extreme 2*

$P_{ex} := \max(P_{ex_g}, P_{ex_f})$ $\quad \boxed{P_{ex} = 7.375 \cdot kip}$ *Maximum Punching Force in Overhang*

$T_{ex} := \max(T_{ex_g}, T_{ex_f})$ $\quad \boxed{T_{ex} = 8.24 \cdot \frac{kip}{ft}}$ *Maximum Axial Tension in Overhang*

Check Main Reinforcement:

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4ksi \\ \max\left[0.85 - 0.05\cdot\left(\frac{f'_c - 4ksi}{ksi}\right), 0.65\right] & \text{otherwise} \end{cases} \qquad \beta_1 = 0.85$$

Stress Block to N.A. Parameter [AASHTO 5.7.2.2]

$$A_{s_prov} := \frac{A_{bar.top_prov}}{S_{bar.top_prov}} = 0.614\cdot\frac{in^2}{ft}$$

Provided Tensile Reinforcement from the deck per unit width

$$A_{s_add} := \frac{A_{bar.top_add}}{S_{bar.top_add}} = 0\cdot\frac{in^2}{ft}$$

Additional Tensile Reinforcement per unit width

$$A_s := A_{s_prov} + A_{s_add} = 0.614\cdot\frac{in^2}{ft}$$

Total Tensile Reinforcement per unit width

$$A'_s := \frac{A_{bar.bot_prov}}{S_{bar.bot_prov}} + \frac{A_{bar.bot_add}}{S_{bar.bot_prov}} = 1.227\cdot\frac{in^2}{ft}$$

Total Compression Reinforcement

Guess $c_0 := 2.0in$ *Guess at Neutral Axis*

Given

$$A_s\cdot f_y = -A'_s\cdot E_s\cdot\left[\frac{0.003}{c_0}\cdot(d' - c_0)\right] + 0.85\cdot f'_c\cdot\beta_1\cdot c_0$$

T = C, Assuming 'Tension' steel yields and 'Compression' steel is also acting in tension.

$c := \text{Find}(c_0)$ $\qquad c = 1.56\cdot in$ *Distance from Extreme Comp. Fiber to N.A.*

$a := \beta_1\cdot c$ $\qquad a = 1.326\cdot in$ *Equivalent Stress Block Depth*

$\varepsilon_s := \frac{0.003}{c}\cdot(d - c)$ $\qquad \varepsilon_s = 0.0079$ *Strain in 'Tension' Steel at Concrete Crushing*

$f'_s := E_s\cdot\left[\frac{0.003}{c}\cdot(d' - c)\right]$ $\qquad f'_s = 14.084\cdot ksi$ *Stress in 'Compression' Reinforcement (Positive is Tension force)*

$$\phi := \begin{cases} 0.90 & \text{if } \varepsilon_s > 0.005 \\ \max\left[0.75, 0.90 - (0.005 - \varepsilon_s)\cdot\frac{0.90 - 0.75}{0.005 - 0.002}\right] & \text{otherwise} \end{cases} \qquad \phi = 0.9$$

Resistance Factor for Concrete in Bending [AASHTO 5.5.4.2]

$$M_n := \left[-A'_s\cdot f'_s\cdot(d - d') + 0.85\cdot f'_c\cdot a\cdot\left(d - \frac{a}{2}\right)\right] \qquad M_n = 17.071\cdot\frac{kip\cdot ft}{ft}$$

Nominal Resistance [AASHTO 5.7.3.2.1]

$\gamma_1 := 1.6$ $\qquad \gamma_3 := 0.67$ *Cracking Moment Coefficients [5.7.3.3.2]*

$$M_{cr} := \frac{\gamma_1\cdot\gamma_3\cdot f_r\cdot t_{slab}^2}{6} \qquad M_{cr} = 6.196\cdot\frac{kip\cdot ft}{ft}$$

Cracking Moment [5.7.3.3.2-1]

$$Check_1 := \text{if}\left(\phi\cdot M_n \ge M_u \wedge M_n \ge M_{ex}, \text{"Overhang Strength OK"}, \text{"Overhang Strength No Good"}\right)$$

$Check_1 =$ "Overhang Strength OK"

$$Check_2 := \text{if}\left[\left(\phi\cdot M_n \ge M_{cr}\right) \vee \left(\phi\cdot M_n \ge 1.33M_u\right), \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No Good"}\right]$$

$Check_2 =$ "Min. Reinforcement is OK" *[AASHTO 5.7.3.3.2]*

Crack Control Check:

$n := \frac{E_s}{E_c} = 7.308$ *Modular Ratio*

$p := \frac{A_s}{d} = 0.009$ *Reinforcement Ratio*

$p \cdot n = 0.066$

$k := \sqrt{2 \cdot p \cdot n + (p \cdot n)^2} - p \cdot n = 0.303$

$j := 1 - \frac{k}{3} = 0.899$

$M_s = 0.748 \cdot \frac{kip \cdot ft}{ft}$ *Service Moment*

$f_{ss} := \frac{M_s}{A_s \cdot j \cdot d} = 2.86 \cdot ksi$ *Tensile stress in steel reinforcement at service limit state*

Ref: MacGregor, J.G., 2009,"Reinforced Concrete: Mechanics and Design, 5th Edition"

$d_c := t_{slab} - d + .5in$ $d_c = 3.312 \cdot in$ *Distance to CG Steel*

$\beta_s := 1 + \frac{d_c}{0.70 \cdot (t_{slab} + .5in - d_c)}$ $\beta_s = 1.832$ *Crack Parameter [AASHTO 5.7.3.4]*

$\gamma_e := 1.0$ *Class 1 Exposure = 1.0*
Class 2 Exposure = 0.75

$s_{min} := \frac{700 \cdot \frac{kip}{in} \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ $s_{min} = 126.982 \cdot in$ *Spacing of mild steel reinforcement (AASHTO 5.7.3.4-1)*

$Check_3 := if\left(\frac{1}{\frac{1}{S_{bar.top_prov}} + \frac{1}{S_{bar.top_add}}} < s_{min}, \text{"Reinforcement Spacing is OK"}, \text{"Reinforcement Spacing is No Good"}\right)$

$Check_3 = \text{"Reinforcement Spacing is OK"}$

Tension Check:

$\phi_t := 0.90$ *Tension Resistance Factor [AASHTO 5.5.4.2]*

$T_n := f_y \cdot (A_s + A'_s)$ $T_n = 110.447 \cdot \frac{kip}{ft}$ *Factored Tensile Resistance [AASHTO 5.7.6.1-1]*

$DC_{ratio} := \frac{T_{ex}}{T_n} + \frac{M_{ex}}{M_n}$ $DC_{ratio} = 0.88$ *Demand to Capacity Ratio - This is an extreme event therefore phi = 1.0 for bending and tension.*

$Check_4 := if(DC_{ratio} < 1.0, \text{"Combined Tension and Moment is OK"}, \text{"Combined Moment and Tension is No Good"})$

$Check_4 = \text{"Combined Tension and Moment is OK"}$

Determine Development Details for Reinforcement:

Overhang Side

$A_{bar} := A_{bar.top_prov} + A_{bar.top_add}$ $\qquad A_{bar} = 0.307 \cdot in^2$

$D_{bar_prov} := D_{bar.top_prov}$ $\qquad D_{bar_prov} = 0.625 \cdot in$

$D_{bar_add} := D_{bar.top_add}$ $\qquad D_{bar_add} = 0 \cdot in$

$$S_{top} := \frac{1}{\frac{1}{S_{bar.top_prov}} + \frac{1}{S_{bar.top_add}}} \qquad S_{top} = 6 \cdot in$$

$\lambda := 1.0$ *Concrete density modification factor (AASHTO 5.4.2.8)*

$\lambda_{rl_top} := 1.0$ *Reinforcement location factor [AASHTO 5.11.2.1.2]*

$$\lambda_{cf_top} := \text{if}\left(2.5in \le 3 \cdot \max\left(D_{bar_prov}, D_{bar_add}\right) \vee S_{top} \le 6 \cdot \max\left(D_{bar_prov}, D_{bar_add}\right), 1.5, 1.2\right) = 1.2$$

Reinforcement coating factor [AASHTO 5.11.2.1.2]

$$c_b := \min\left(\frac{S_{top}}{2}, 2.5in + \frac{\max\left(D_{bar_prov}, D_{bar_add}\right)}{2}\right) = 2.813 \cdot in$$

Bar cover

$$\lambda_{rc_top} := \max\left(\min\left(\frac{\max\left(D_{bar_prov}, D_{bar_add}\right)}{c_b + 0}, 1\right), 0.4\right) = 0.4$$

Reinforcement confinement factor [AASHTO 5.11.2.1.3-1]

$$\lambda_{er_top} := \max\left[\frac{M_{ex}}{M_n}, \frac{M_{cr}}{\left(\phi \cdot M_n\right)}, 1.33\frac{M_u}{\left(\phi \cdot M_n\right)}, \frac{S_{top}}{s_{min}}, DC_{ratio}\right] = 0.88$$

Excess reinforcement factor ($A_{s\ required}/A_{s\ provided}$), [AASHTO 5.11.2.1.3]

$$L_{db_prov} := 2.4 \cdot D_{bar_prov} \cdot \frac{f_y}{\sqrt{f'_c \cdot ksi}} = 3.75\ ft$$

Basic development length of the provided bars from the deck in tension [AASHTO 5.11.2.1.1-2]

$$L_{d_prov} := L_{db_prov} \cdot \frac{\min\left(\lambda_{rl_top} \cdot \lambda_{cf_top}, 1.7\right) \cdot \lambda_{rc_top} \cdot \lambda_{er_top}}{\lambda}$$

$L_{d_prov} = 1.583\ ft$ *Basic Development Length of the provided bars from the deck in Tension [AASHTO 5.11.2.1.1] - with Epoxy Coated modification factor*

$L_{spl_prov} := 1.3 \cdot \left(L_{d_prov}\right)$ $\qquad L_{spl_prov} = 2.0585\ ft$ *Lap Splice of the provided bars from the deck in Tension Region [AASHTO 5.11.2.1.1 & 5.11.5.3.1, Class C Splice Length]*

$$L_{dh_prov} := \max\left[(1.2)\frac{38 \cdot D_{bar_prov} \cdot f_y}{60 \cdot \sqrt{f'_c \cdot ksi}}, 6in, 8 \cdot D_{bar_prov}\right]$$

$L_{dh_prov} = 1.188\ ft$ *Development Length of the provided bars from the deck in Tension with a Standard Hook [AASHTO 5.11.2.4.1] - with Epoxy Coated modification Factor*

$L_{db_add} := 2.4 \cdot D_{bar_add} \cdot \frac{f_y}{\sqrt{f'_c \cdot ksi}} = 0$

Basic development length of the provided bars from the deck in tension [AASHTO 5.11.2.1.1-2]

$L_{d_add} := L_{db_add} \cdot \frac{\min(\lambda_{rl_top} \cdot \lambda_{cf_top}, 1.7) \cdot \lambda_{rc_top} \cdot \lambda_{er_top}}{\lambda}$

$L_{d_add} = 0$

Basic Development Length of the additional bars from the overhang in Tension [AASHTO 5.11.2.1.1] - with Epoxy Coated modification factor

$L_{spl_add} := 1.3 \cdot (L_{d_add})$

$L_{spl_add} = 0$

Lap Splice of the additional bars from the overhang in Tension Region [AASHTO 5.11.2.1.1 & 5.11.5.3.1, Class C Splice Length]

$L_{dh_add} := \max\left[(1.2)\frac{38 \cdot D_{bar_add} \cdot f_y}{60 \cdot \sqrt{f'_c \cdot ksi}}, 6in, 8 \cdot D_{bar_add}\right]$

$L_{dh_add} = 0.5\,ft$

Development Length of the additional bars from the overhang in Tension with a Standard Hook [AASHTO 5.11.2.4.1] - with Epoxy Coated modification Factor

$L_{all} := W_{curb} - 2in$

$L_{all} = 1.833\,ft$

Length Available for Bar Development for Full Capacity

Bars Provided from the transverse deck reinforcement

$Bar_Detail_prov := if(L_{all} > L_{d_prov}, \text{"No Hook Req'd"}, if(L_{all} > L_{dh_prov}, \text{"Use Standard 180deg Hook"}, \text{"Add'l Inv Req'd"}))$

$Bar_Detail_prov = \text{"No Hook Req'd"}$

Additional bars added to the overhang

$Bar_Detail_add := if(L_{all} > L_{d_add}, \text{"No Hook Req'd"}, if(L_{all} > L_{dh_add}, \text{"Use Standard 180deg Hook"}, \text{"Add'l Inv Req'd"}))$

$Bar_Detail_add = \text{"No Hook Req'd"}$

Check Shear Resistance:

One-Way Shear per AASHTO 5.8.3.3

$$d_v := \max\left(d - \frac{a}{2}, 0.90 \cdot d, 0.72 \cdot t_{slab}\right) = 6.12 \cdot in$$

Effective shear Depth (5.8.2.9)

$$V_u := \gamma_{LL.str} \cdot V_{LL3.f} + \gamma_{DC.str} \cdot \left[w_{rail} + \gamma_{conc} \cdot (t_{slab} + t_{curb} + .5in) \cdot (W_{curb}) \ldots + \gamma_{conc} \cdot (t_{slab} + .5in) \cdot \left(L_{over} - \frac{t_w}{2} - W_{curb}\right) \right] \ldots + \gamma_{DW.str} \cdot \left[\gamma_{asph} \cdot (t_{asph}) \cdot \left(L_{over} - \frac{t_w}{2} - W_{curb}\right) \right] = 0.662 \cdot \frac{kip}{ft}$$

Ultimate shear force

$$s_{xe} := \min\left[\max\left[\left[\min\left[(d - d'), d_v\right] \cdot \frac{1.38}{0.75 + 0.63}\right], 12in\right], 80in\right] = 1\,ft$$

Shear parameter (5.8.3.4.2-5)

$$\varepsilon_s := \frac{\frac{\max(M_{u_f}, V_u \cdot d_v)}{d_v} + V_u}{E_s \cdot A_s} = 0$$

Strain in steel at ultimate shear (5.8.3.4.2-4)

$$\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} \cdot \frac{51}{39 + \frac{s_{xe}}{in}} = 4.337$$

AASHTO 5.8.3.4.1

$$V_c := \min\left(0.0316 \cdot \beta \cdot \sqrt{f'_c \cdot ksi} \cdot d_v, 0.25 f'_c \cdot d_v\right) = 20.129 \cdot \frac{kip}{ft}$$

AASHTO 5.8.3.3-3

$$\phi_v := 0.90$$

AASHTO 5.5.4.2.1

$$Check_5 := \text{if}\left(\phi_v \cdot V_c > V_u, \text{"Shear is OK"}, \text{"Additional Investigation Required"}\right)$$

AASHTO 5.8.2.4-1

$$\boxed{Check_5 = \text{"Shear is OK"}}$$

Punching Shear per AASHTO A13.4.3.2

$$t_{slab} = 8.5 \cdot in$$

Shear depth for punching shear

$$L_{V.Perim} = 6.954\,ft$$

Effective punching shear perimeter

$$v_c := \min\left[\left[\left[0.0633 + \frac{0.1265}{\left(\frac{W_b}{d_b}\right)}\right] \cdot \sqrt{f'_c \cdot ksi}\right], \left(0.1265 \cdot \sqrt{f'_c \cdot ksi}\right)\right] = 0.253 \cdot ksi$$

Nominal Shear Resistance Provided by Tensile Stresses in the Concrete

$$V_n := t_{slab} \cdot L_{V.Perim} \cdot v_c \qquad V_n = 179.45 \cdot kip$$

Nominal Shear Resistance of the Section Considered

$$\phi_{v.Punching.Shear} := 1.0$$

Resistance Factor [AASHTO A13.4.3.2]

$$V_u := P_{ex} \qquad V_u = 7.375 \cdot kip$$

$$Check_6 := \text{if}\left(\phi_{v.Punching.Shear} \cdot V_n > V_u, \text{"Punching Shear is OK"}, \text{"Additional Investigation Required"}\right)$$

$$\boxed{Check_6 = \text{"Punching Shear is OK"}}$$

Interior Positive Reinforcement Deck Design

Methodology:

1) Use 8.5 inch deck thickness and 3.0 inch future wearing surface
2) Negative reinforcement was designed along with the precast deck panels, only positive reinforcmeent at the ends of the deck will be analyzed below

Properties:

$f'_c := 4.0\text{ksi}$ — *Concrete Strength*

$F_y := 60\text{ksi}$ — *Yield Stress of Mild Reinforcing Steel*

$f_r := 0.24\sqrt{f'_c \cdot \text{ksi}}$ $\quad f_r = 0.48 \cdot \text{ksi}$ — *Rupture Stress of Concrete [AASHTO 5.4.2.6]*

$E_s := 29000\text{ksi}$ — *Modulus of Elasticity of Steel*

$E_c := 2500\text{ksi}\left(\frac{f'_c}{\text{ksi}}\right)^{\frac{1}{3}}$ $\quad E_c = 3969 \cdot \text{ksi}$ — *Modulus of Elasticity of Concrete [AASHTO C5.4.2.4-1]*

$W_{conc} := 150\text{pcf}$ — *Unit weight of Concrete*

$D_{bar.top} := 0.625\text{in}$ — *Reinforcing Bar Diameter*

$A_{bar.top} := \pi D_{bar.top}^2 \div 4 = 0.307 \cdot \text{in}^2$ — *Reinforcing Bar Area*

$D_{bar.bot} := 0.625\text{in}$ — *Reinforcing Bar Diameter*

$A_{bar.bot} := \pi D_{bar.bot}^2 \div 4 = 0.307 \cdot \text{in}^2$ — *Reinforcing Bar Area*

Geometry:

$\text{skew} := 0\text{deg}$ — *Skew angle of reinforcement with respect to girder direction*

$S_{gird} := 7.5\text{ft}$ — *Girder Spacing, Normal to Girders*

$t_{slab} := 8.5\text{in}$ — *Thickness of Deck, structural component*

$t_w := 0.75\text{in}$ — *Web Thickness of Girder*

$b_f := 20\text{in}$ — *Width of Top Flange*

$S_{bar.main} := 6\text{in} \div \cos(\text{skew}) = 6 \cdot \text{in}$ — *Main Reinforcement Bar Spacing, perpendicular to design direction*

$S_{bar.bot} := 9\text{in}$ — *Bar Spacing on Bot of Slab, Parallel with Traffic*

$W_{slab} := 35\text{ft}$ — *Overall Bridge Width*

Geometry, cont:

$A_{bar_long} := (0.625in)^2 \cdot \frac{\pi}{4}$ $\quad A_{bar_long} = 0.31 \cdot in^2$ *Bar area for longitudinal distribution bars*

$d' := 2.5in + D_{bar.top} \div 2$ $\quad d' = 2.813 \cdot in$ *Distance to Top Layer of Main Reinforcement*

$d := t_{slab} - 1.5in - (D_{bar.bot} \div 2)$ $\quad d = 6.687 \cdot in$ *Distance to Bot Layer of Main Reinforcement*

$L_{eff} := (S_{gird} - 1.0t_w) \div \cos(skew)$ $\quad L_{eff} = 7.438\ ft$ *Effective Deck Length [AASHTO 9.7.2.3]*

Loads & Moments:

Dead Loads:

$DW := (3in) \cdot 0.14 \frac{kip}{ft^3} + 0.5in \cdot .150 \cdot \frac{kip}{ft^3}$ $\quad DW = 41.25 \cdot \frac{plf}{ft}$ *Uniform Load of Wearing Surface and Membrane*

$DC := W_{conc} \cdot t_{slab}$ $\quad DC = 106.25 \cdot \frac{plf}{ft}$ *Uniform Load due to Self Weight*

Base moments on continuous 4 span girder model with evenly distributed load

$M_{DW.pos} := 0.0772DW \cdot L_{eff}^2$ $\quad \boxed{M_{DW.pos} = 0.176 \cdot \frac{kip \cdot ft}{ft}}$ *Max Positive Moment due to DW*

$M_{DC.pos} := 0.0772DC \cdot L_{eff}^2$ $\quad \boxed{M_{DC.pos} = 0.454 \cdot \frac{kip \cdot ft}{ft}}$ *Max Positive Moment due to DC*

Live Loads:

- *AASHTO Table A4-1*

$M_{LL.pos} := 5.44 \frac{kip \cdot ft}{ft}$ $\quad \boxed{M_{LL.pos} = 5.44 \cdot kip \cdot \frac{ft}{ft}}$ *Max Positive LL moment with Impact and Multiple Presence Included*

Loads & Moments, cont:

Strength I Design Moment:

$\eta := 1.0$ *Load modifier for Ductility, Importance, and Redundancy*

$\gamma_{DC} := 1.25$ *Load Factor for Dead Load - Components*

$\gamma_{DW} := 1.50$ *Load Factor for Dead Load - Wearing Surface*

$\gamma_{LL} := 1.75$ *Load Factor for Live Load*

$$M_{u.pos} := \eta \cdot \left(\gamma_{DC} \cdot M_{DC.pos} + \gamma_{DW} \cdot M_{DW.pos} + \gamma_{LL} \cdot M_{LL.pos}\right)$$

$$\boxed{M_{u.pos} = 10.351 \cdot kip \cdot \frac{ft}{ft}}$$ *Strength I Moment, Ultimate Positive*

Service I Design Moment:

$\eta := 1.0$ *Load modifier for Ductility, Importance, and Redundancy*

$\gamma_{DC.s} := 1.0$ *Load Factor for Dead Load - Components*

$\gamma_{DW.s} := 1.0$ *Load Factor for Dead Load - Wearing Surface*

$\gamma_{LL.s} := 1.0$ *Load Factor for Live Load*

$$M_{s.pos} := \eta \cdot \left(\gamma_{DC.s} \cdot M_{DC.pos} + \gamma_{DW.s} \cdot M_{DW.pos} + \gamma_{LL.s} \cdot M_{LL.pos}\right)$$

$$\boxed{M_{s.pos} = 6.07 \cdot \frac{kip \cdot ft}{ft}}$$ *Service I Moment, Ultimate Positive*

Check Main Reinforcement (Positive Moment):

$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4ksi \\ \max\left[0.85 - 0.05\cdot\left(\frac{f'_c - 4ksi}{ksi}\right), 0.65\right] & \text{otherwise} \end{cases}$ $\quad \beta_1 = 0.85$ *Stress Block to N.A. Parameter [AASHTO 5.7.2.2]*

$A_s := \frac{A_{bar.bot}}{S_{bar.main}}$ $\quad A_s = 0.614\cdot\frac{in^2}{ft}$ *Tensile Reinforcement per unit width*

$A'_s := \frac{A_{bar.top}}{S_{bar.main}}$ $\quad A'_s = 0.614\cdot\frac{in^2}{ft}$ *Compression Reinforcement*

Guess $\quad c_0 := 2.0in$ *Guess at Neutral Axis*

Given

$A_s\cdot F_y = -A'_s\cdot\min\left[E_s\cdot\left[\frac{0.003}{c_0}\cdot(d' - c_0)\right], 60ksi\right] + 0.85\cdot f'_c\cdot\beta_1\cdot c_0$ *T = C, Assuming 'Tension' steel yields and 'Compression' steel is also acting in tension.*

$c := \text{Find}(c_0)$ $\quad c = 1.855\cdot in$ *Distance from Extreme Comp. Fiber to N.A.*

$a := \beta_1\cdot c$ $\quad a = 1.577\cdot in$ *Equivalent Stress Block Depth*

$\varepsilon_s := \frac{0.003}{c}\cdot(d - c)$ $\quad \varepsilon_s = 0.0078$ *Strain in 'Tension' Steel at Concrete Crushing*

$F'_s := \min\left[E_s\cdot\left[\frac{0.003}{c}\cdot(d' - c)\right], 60ksi\right]$ $\quad F'_s = 44.872\cdot ksi$ *Stress in 'Compression' Reinforcement (Positive is Tension force)*

$\phi := \begin{cases} 0.90 & \text{if } \varepsilon_s > 0.005 \\ \max\left[0.75, 0.90 - (0.005 - \varepsilon_s)\cdot\frac{0.90 - 0.75}{0.005 - 0.002}\right] & \text{otherwise} \end{cases}$ $\quad \phi = 0.9$ *Resistance Factor for Concrete in Bending [AASHTO 5.5.4.2]*

$\phi M_n := \phi\cdot\left[A_s\cdot F_y\cdot\left(d - \frac{a}{2}\right) + A'_s\cdot F'_s\cdot\left(d' - \frac{a}{2}\right)\right]$

$\phi M_n = 20.467\cdot\frac{kip\cdot ft}{ft}$ *Factored Resistance [AASHTO 5.7.3.2.1]*

$\gamma_1 := 1.6$ $\quad \gamma_3 := 0.67$ *Cracking Moment Coefficients [5.7.3.3.2]*

$M_{cr} := \frac{\gamma_1\cdot\gamma_3\cdot f_r\cdot t_{slab}^2}{6}$ $\quad M_{cr} = 6.196\cdot\frac{kip\cdot ft}{ft}$ *Cracking Moment [5.7.3.3.2-1]*

$M_{u.pos} = 10.351\cdot\frac{kip\cdot ft}{ft}$ *Ultimate Moment, Positive*

$Check_1 := \text{if}\left(\phi M_n \ge M_{u.pos}, \text{"Deck is OK for Strength"}, \text{"Deck is No Good for Strength"}\right)$

$Check_1 = \text{"Deck is OK for Strength"}$

$Check_2 := \text{if}\left[\left(\phi M_n \ge M_{cr}\right) \vee \left(\phi M_n \ge 1.33M_{u.pos}\right), \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No Good"}\right]$

$Check_2 = \text{"Min. Reinforcement is OK"}$ *[AASHTO 5.7.3.3.2]*

Crack Control Check (Positive Moment):

$n := \frac{E_s}{E_c} = 7.308$ *Modular Ratio*

$p := \frac{A_s}{d} = 0.008$ *Reinforcement Ratio*

$p \cdot n = 0.056$

$k := \sqrt{2 \cdot p \cdot n + (p \cdot n)^2} - p \cdot n = 0.283$

$j := 1 - \frac{k}{3} = 0.906$

$M_{s.pos} = 6.07 \cdot \frac{kip \cdot ft}{ft}$ *Service Moment*

$f_{ss} := \frac{M_{s.pos}}{A_s \cdot j \cdot d} = 19.6 \cdot ksi$ *Tensile stress in steel reinforcement at service limit state*

Ref: MacGregor, J.G., 2009,"Reinforced Concrete: Mechanics and Design, 5th Edition"

$d_c := t_{slab} - d$ $d_c = 1.813 \cdot in$ *Distance to CG Steel*

$\beta_s := 1 + \frac{d_c}{0.70 \cdot (t_{slab} - d_c)}$ $\beta_s = 1.387$ *Crack Parameter [AASHTO 5.7.3.4]*

$\gamma_e := 1.$ *Class 1 Exposure = 1.0*
Class 2 Exposure = 0.75

$s := \frac{700 \cdot \frac{kip}{in} \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ $s = 22.121 \cdot in$ $S_{bar.main} = 6 \cdot in$

$Check_3 := \text{if}(S_{bar.main} < s, \text{"Reinforcement Spacing is OK"}, \text{"Reinforcement Spacing is No Good"})$

$Check_3 = \text{"Reinforcement Spacing is OK"}$

Steel Reinforcement Distribution:

$A_{s.bot} := \frac{A_{bar_long}}{S_{bar.bot}}$ $\quad A_{s.bot} = 0.409 \cdot \frac{in^2}{ft}$ *Steel Reinforcement per Unit Width, Parallel to Traffic, bottom layer*

$A_{min.bot} := \min\left(2.2 \div \sqrt{\frac{L_{eff}}{ft}} \cdot \frac{A_{bar.top}}{S_{bar.main}}, \frac{2}{3} \frac{A_{bar.top}}{S_{bar.main}}\right)$ $\quad A_{min.bot} = 0.409 \cdot \frac{in^2}{ft}$ *[AASHTO 9.7.3.2]*

$Check_7 := \text{if}\left(A_{s.bot} \geq A_{min.bot}, \text{"Secondary Bottom Reinforcement OK"}, \text{"Secondary Bottom Reinforcement No Good"}\right)$

$Check_7 = \text{"Secondary Bottom Reinforcement No Good"}$

Ok, required and provided are the same just the rounding effects the results. The longitudinal spacing is 2/3 of the transverse which is the limit.

Temperature And Shrinkage Steel:

Only checking bottom of slab reinforcing as Top of Deck is checked in the Precast Panel Deck worksheet.

$A_{ts.min} := \min\left[\max\left[0.11 \cdot in^2, \left[\frac{1.30 \cdot W_{slab} \cdot t_{slab}}{2 \cdot (W_{slab} + t_{slab}) \cdot F_y}\right] \cdot \frac{kip}{in}\right], 0.60 in^2\right]$ *Temperature and Shrinkage Steel Boundaries [AASHTO 5.10.8]*

$\frac{A_{ts.min}}{S_{gird}} = 0\ ft$ $\quad A_{ts.min} = 0.11 \cdot in^2$

$A_{s.bot} := \frac{A_{bar_long}}{S_{bar.bot}}$ $\quad A_{s.bot} = 0.409 \cdot \frac{in^2}{ft}$ *Reinforcement Provided in Top of Deck, Parallel with Traffic*

$Check_8 := \text{if}\left(A_{s.bot} > \frac{A_{ts.min}}{S_{gird}}, \text{"Temperature and Shrinkage Steel OK"}, \text{"Temperature and Shrinkage Steel No Good"}\right)$

$Check_8 = \text{"Temperature and Shrinkage Steel OK"}$

Determine Development Details for Reinforcement:

Only need to look at bottom reinforcement, top reinforcement are full bridge width and detailed in the Deck Overhang worksheet.

$D_{bar.bot} = 0.625 \cdot in$ — *Diameter of bottom reinforcing bars*

$\lambda := 1.0$ — *Concrete density modification factor (AASHTO 5.4.2.8)*

$\lambda_{rl_top} := 1.0$ — *Reinforcement location factor [AASHTO 5.11.2.1.2]*

$\lambda_{cf_top} := \text{if}\left(1.5in \le 3 \cdot D_{bar.bot} \vee S_{bar.bot} \le 7 \cdot D_{bar.bot}, 1.5, 1.2\right) = 1.5$ — *Reinforcement coating factor [AASHTO 5.11.2.1.2]*

$c_b := \min\left(\frac{S_{bar.bot}}{2}, 1.5in + \frac{D_{bar.bot}}{2}\right) = 1.812 \cdot in$ — *Bar cover*

$\lambda_{rc_top} := \max\left(\min\left(\frac{D_{bar.bot}}{c_b + 0}, 1\right), 0.4\right) = 0.4$ — *Reinforcement confinement factor [AASHTO 5.11.2.1.3-1]*

$\lambda_{er_top} := 1$ — *Excess reinforcement factor ($A_{s\ required}/A_{s\ provided}$), conservatively taken as 1 [AASHTO 5.11.2.1.3]*

$L_{db} := 2.4 \cdot D_{bar.bot} \cdot \frac{F_y}{\sqrt{f'_c \cdot ksi}} = 3.75\ ft$ — *Basic development length [AASHTO 5.11.2.1.1-2]*

$L_d := L_{db} \cdot \frac{\min\left(\lambda_{rl_top} \cdot \lambda_{cf_top}, 1.7\right) \cdot \lambda_{rc_top} \cdot \lambda_{er_top}}{\lambda}$ — *Development Length [AASHTO 5.11.2.1.1]*

$L_d = 2.25\ ft$

$L_{dh} := \max\left[(1.2)\frac{38 \cdot D_{bar.bot} \cdot F_y}{60 \cdot \sqrt{f'_c \cdot ksi}}, 6in, 8 \cdot D_{bar.bot}\right]$ — *Development Length of the provided bars from the deck in Tension with a Standard Hook [AASHTO 5.11.2.4.1] - with Epoxy Coated modification Factor*

$L_{dh} = 1.188\ ft$

$L_{all} := b_f - 4in = 1.333\ ft$ — $L_{all} = 1.333\ ft$ — *Length Available for Bar Development for Full Capacity*

$Bar_Detail_prov := \text{if}\left(L_{all} > L_d, \text{"No Hook Req'd"}, \text{if}\left(L_{all} > L_{dh}, \text{"Use Standard 180deg Hook"}, \text{"Add'l Inv Req'd"}\right)\right)$

$Bar_Detail_prov = \text{"Use Standard 180deg Hook"}$

The loads that will require the fully developed secton will only occur at midspan of the section with the loads at the tips of the girder flanges being minimal. Based on inspection, straight bars will be adequate as they will have enough capacity to carry the small loads at the flange tips and will also fully develop by midsection of the beam where the maximum moments will occur.

T·Y·LIN INTERNATIONAL

Project: Waitsfield | Job No. 411847.00 | Sheet: 1 of 6

Item: Deck Reinforcement Details | Designer: TJP | Date: 12/11/15

Checker: BLT / JKO | Date: 12/15/15 / 2/1/16

Grid: 1/10"

S5001.Z - Top of Deck - longitudinal

Qty - $\frac{172' + 2'6''/\cos(45°) + 1'1''/\cos(45°) + 2'4'' - 2(3'')}{(40 - 2'4'')}$

$= 4.75 \Rightarrow$ 5 bars per row

Lap = 2'4"

(use the laplength from Overhang Design worksheet w/o reduction of excess reinforcement)

$\frac{35' - 2(3'') - 0.625''}{6''} = 68.9 \rightarrow 69$ spaces $= 70$ rows $+ 6 - 2 = 82$

(6 for curbs and bottom of deck in overhangs each side)

Total = 5 bars / row · 82 rows = $\boxed{410 \text{ bars}}$

Length - $\frac{172' + 2'6''/\cos(45°) + 1'1''/\cos(45°) + 2'4'' \cdot 4 - 2(3'')}{5}$ = 37.18' $\Rightarrow$ $\boxed{37' \ 3''}$

S5002.Z - Top of Deck - Transverse - full width

Qty - $\frac{172' + 2'6''/\cos(45°) + 1'1''/\cos(45°) - 35' - 2(3'')}{6''}$ = 283.14 = 284 spaces

= $\boxed{285 \text{ Bars}}$

Length 35' - 2(3") = $\boxed{34' \ 6''}$

S5003.Z - Top of Deck - 1st transverse in skewed portion

Qty - $\boxed{2 \text{ Bars}}$ (1 each end)

Length - $35' - 3'' - (6'' - 3'') - 3''/\cos(45)$ =

(6" space, - 2" from last S5002.Z to end of deck

= 34.146 $\rightarrow$ $\boxed{34' \ 1''}$

S5004.Z - S5067.Z

Qty - $\boxed{2 \text{ Bars}}$ each

Length Increment = $\boxed{6''}$

Length S5004.Z = 34'1" - 6" = 33'7"

Length S5067.Z = 2'1"

Determine how many bars to maintain the bar > 2'0" long.

$\frac{33'7'' - 2'1''}{6''}$ = 63 spa = 64 sets

T·Y·LIN INTERNATIONAL

Project: Waitsfield	Job No. 411847.00	Sheet: 2 of 6	
Item: Deck Reinforcement Details	Designer: TJP	Date: 12/14/15	
	Checker: BLT	Date: 12/15/15	
		Grid: 1/10"	

S5068.2 - End of Deck bars along joint

Qty - 2 top of deck - 2 bottom of deck each end

= 2 (2+2) = 8 Bars

Length = 48'7" → measured in CAD based on outside bar, accounts fo 6" chamfer in acute corner of deck

S5069.2 - S5078.2 - End of Deck - Bottom, longitudinal

Qty - 1 Bar per panel · 4 panels · 2 sides = 8 bars per type

Spacing of bars = 9" = increment of bars

# of bars in set = $\frac{6'6''}{9''}$ = 8.66 → 9 spa → 10 bar sizes

Total width of bar set to keep 9" spacing = 6'9"

$\text{Length}_{S5069.2} = 8'3'' + 2 \cdot 5'/\cos(45) + \frac{(6'9'' - 6'6'')}{2} - 3''/\sin(45)$

= 11.55 → 11'6"

Note: Distance from edge of panel to end of deck is the same on both sides of the bridge

$\text{Length}_{S5078.2} = 1'9'' + 2 \cdot 5'/\cos(45) - \frac{(6'9'' - 6'6'')}{2} - 3''/\sin(45)$

= 4.81 = 4'9"

S5079.2 - - End of Deck - Bottom - transverse → obtuse corner before set A

Qty - 5 bars EA. End = 10 Bars

Length - 7'6" + 2'6" + $\frac{20''}{2}$ - 4" - 3" = 10'3"

S5080.2 - S5089.2 - End of Deck - Bottom - Transverse - Set A

Qty - 1 Bar each end = 2 Bars

$\text{Length}_{S5080.2}$ - 7'6" + 2'6" + $\frac{20''}{2}$ - 4" - 3"/sin45 - 6" = 9.65 → 9'7"

INCREMENT - 6"

$\text{Length}_{S5089.2}$ = 9'7" - 9.6" = 5.083 → 5'1"

# of Bars in Set A

$= \frac{8'-6''-6''-3''}{6''} - \left(1'9'' + 2.5'/\cos(45) - \frac{(2.5'+6'')}{\tan 45}\right)$

= 8.9 → 9 spaces = 10 Bars in set A

S5090.2 - S5104.2 - End of Deck - Bottom - Transverse - Set B

Qty - 2 sets each end = 14 Bars

Number of Bars in Set B

$$= \frac{8'-6" - 2(3")}{6"} = 14 \text{ spaces} = 15 \text{ Bars}$$

$$\text{Length}_{S5090.2} = \frac{7'6" + \frac{1'9" + 2'6"}{\cos(45)} - 3"}{\tan 45} - \frac{3"}{\sin 45} = 12.18 \rightarrow 12'2"$$

$$\text{Increment} = \frac{6"}{\tan 45} = 6"$$

$$\text{Length}_{S5104.2} = 12'2" - \frac{(8'-6" - 2(3"))}{\tan(45)} = 5'2"$$

S5105.2 - S5129.2 - End of Deck - Bottom - Transverse - Set C

Qty - 12 Bars (1 each acute corner)

Note: minimum of 2' bar length

Number of Bars in Set

$$= \frac{6'6" + \left(\frac{20"}{2} - 4"\right) + 2'6" + \left(1'9" + 2'6"/\cos 45\right) - 3" - 2' - 3" - 3"/\sin 45}{6"}$$

= 23.86' → Use 24 spaces to have last bar be close to 2' long

= 25 Bars in Set

$$\text{Length}_{S5105.2} = 6'6" + \left(\frac{20"}{2} - 4"\right) + 2'6" + \left(1'9" + 2'6"/\cos(45)\right) - 3" - 3" - 3"/\sin 45$$

$$= 13.93 \Rightarrow 13'11"$$

$$\text{Increment} = 6"$$

$$\text{Length}_{S5129.2} = 13'11" - 24 \cdot 6" = 1'11"$$

T·Y·LIN INTERNATIONAL

Project:	Waitsfield	Job No.	41184700	Sheet:	4 of 6
Item:	Deck Reinforcement	Designer:	TJP	Date:	12/15/15
		Checker:	BLT	Date:	12/15/15
				Grid:	1/10"

S5130.2 - Outside edge of abutment 2 edge beam

Qty - 1 Bar

Length $\frac{4(7'6'') - 0.75'' - 2(1'')}{\cos(45°)} = 42.10'$ (for 1" extra on each side additional cover for fitup) = 42' 1"

S5131..2 - Bottom of Edge Beam - Between Girders (Abutment 2 only)

Qty - 2 Bars per bay · 4 Bays = 8 Bars

Length - $\frac{7'6' - 0.75'' - 2(1'')}{\cos(45)} = 10.28'$ Say 10'3"

S5501.2 - Curb bars

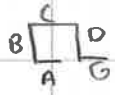
Qty - 7 Bars per post · 25 post per side = 175 bars per side · 2

12" Spacing between Posts -

Interior spaces - $\frac{7'4\frac{1}{2}" - 2(3(6")) - 2(1')}{12"} = 2.375 \rightarrow 3$ spaces = 4 Bars/space

End spans - $\frac{6'3" - 2(3(6")) - 2(1')}{12"} = 1.25 \Rightarrow 2$ Spa = 3 Bars

Total: 175 + (2 + 3)(3 Bars) + 19 (4 Bars) = 266 Bars per side = **532 total**

Dimensions: (S5) 

Note: left side of the bridge has different end spans but leads to less bars needed (88) assumed conservative to keep the other 3 Bars for ends of the deck

A = 10" (standard length)

B = 9" + 7" - 3" - 1½" = 11.5" ⇒ 11"

C = 2' - 2(3") = 1'6"

D = B = 11"

G = A = 10"

Length = **5'0"**

Overall width = 1'6" + 10" = 28"

Distance from face of panel to fascia = 2'6" + $\frac{20"}{2}$ - 4" = 3'0"

Check if bar fits = 3'0" > 3" + 28" = 2'7" → so OK

S5502.2 - Bottom of deck overhang bars

Qty - $\frac{20.8'}{6"}$ = 320 bars · 2 side

= **640 Bars**

(End of precast panel to precast panel)

Dimensions: (1)

A = 7"

B = 2'6" + $\left(\frac{20"}{2} - 4"\right)$ - 3" = 2'9"

G = 0

Length = **3'4"**

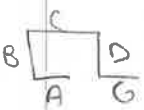
T·Y·LIN INTERNATIONAL

Project:	Waitsfield	Job No.	411847.00	Sheet:	6 of 6
Item:	Deck Reinforcement Plan	Designer:	TJP	Date:	12/15/15
		Checker:	BLT/JKO	Date:	12/15/15 / 2/1/16
				Grid:	1/10"

S5503.2 - End Beam - Abutment 2 -

Qty - $\left(\frac{7'6" - 20"}{9"}\right) = 7.7 = 8 \text{ Spa} \Rightarrow 9$ Bars / Bay · 4 Bays = **36 Bars**

Dimensions - (S5)



A = 10" (standard)

B = 8.5" + 4.25" + 4" - 2.5" - 1.5" = 12.75" - 1" for additional cover for hooked tol = 11"

C = (2'2" - 2(3")) / cos(45) = 2.35" ⇒ 2'4"

D = B = 11"

G = A = 10"

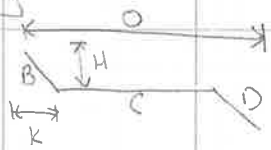
Length = 70" = **5'10"**

S5504.2 - Obtuse Corner Deck Bar

Qty - 1 both top and bottom each corner

= **4 Bars**

Dimensions: (19)



B = C = lap length = 2'4"

$H = K = \sqrt{\left(\frac{2'4"}{2}\right)^2} = 1.65' = 1'8"$

D = O

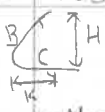
O = K + C = 2'4" + 1'8" = 4'

Length: B + C = **4'8"**

S5505.2 - Acute corner deck Bar

Qty - Same as S5504.2 = **4 Bars**

Dimensions: (27)



B = C = lap length = 2'4"

$H = K = \sqrt{\left(\frac{2'4"}{2}\right)^2} = 1.65 = 1'8"$

Length: **4'8"**

3b. Steel Girder Analysis and Detailing

The change in deck thickness from the original 8” thick design with 3” of asphalt to a 9” thick design and allowance of a 3” future overlay, required changes to the girder camber, dead load deflections and shear connector layouts. The VTrans-proposed girders were input into the commercial software, MDX, which allowed the new values to be calculated based on the change in the deck thickness. Sufficient analyses were performed to identify new deflections and that the final structure had sufficient strength.

Shear connectors were evenly distributed along each girder to match the total quantity and intent of the original design since fatigue shear range was unchanged from the original design and the section properties only nominally changed.

Along with determining the new camber, dead load deflection and shear connector layout, the end crossframe had to be changed to allow for an edge beam in the end of deck at Abutment No. 2. The end crossframe was designed in MathCAD in accordance with AASHTO requirements and the forces were derived from the MDX model created.

The VTrans-proposed girders were also investigated for a future configuration with a new 9” deck, 3” wearing surface, and a 1/16” reduction in plate thickness (1/32” from each face). The investigation yielded the girders have sufficient capacity, including the field splice.

```
WAITSFIELD
Girder 1 : Input File : Definition
Mon Dec 21 09:06:26 2015


ID: WAITSFIELD
CONDITIONS
  CLASS B SURFACE
  ENGLISH INPUT
  ENGLISH OUTPUT
  FINE RESOLUTION OUTPUT
  FLANGE BOLTS NOT STAGGERED
  FULL DEPTH CONNECTION PLATES
  INCLUDE SLAB FILLER IN SECTION PROPERTIES
  INTERMEDIATE TRANSVERSE STIFFENERS ONE SIDE OF WEB
  LRFD METHOD
  M270-50W BOTTOM FLANGE STEEL
  M270-50W STIFFENER STEEL
  M270-50W TOP FLANGE STEEL
  M270-50W WEB STEEL
  RATE MODE
  SINGLE BEARING STIFFENERS EACH SIDE
  SYSTEM FORCES
DATA
  ADTT 245
  BFISPB 10.5
  BFISPT 1.
  BFNHOLES 6
  BFOSPB 24.
  BFOSPT 1.
  BR 10.25 18. 18. 18. 18. 18. 18. 18. 18. 17.75
  BSPL 45. 82.
  EDGEH 1.5
  EDGEV 1.5
  EDGEW 2.125
  ESLABW 75.
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FBLTDIAM  0.875
  FILLET 1.
  FPC 4.
  FPEDGE 1.5
  FPEND 1.5
  FSPBSP 3.
  GAGEBF 3.75
  GAGEBFW 3.
  GAGEH 3.
  GAGETF 5.75
  GAGETFW 2.75
  GAGEV 3.5
  IGIRD 1
  LIFE 100
  NBLTB 48
  NBLTT 32
  NLINEH 15
  NLINEV 4
  NSTUDL 2
  SLABT 9.
  SLABWEAR  0.5
  SPL 122.
  SPLBFT 1.25 1.75 1.25
  SPLBFW 24.
  SPLT  0.5
  SPLTFT 1.25 1.5 1.25
  SPLTFW 20.
  SPLTST  0.5
  SPLTSW 9.5
  SPLTYP 1
  SPLWD 60.
  SPLWT  0.75
  SPN 172.
  SS 1.
  STD  0.875
  STFGAP  0.
  STH 7.
  SUPBST 1.
  SUPBSW 9.5
  TFISPB 8.75
  TFISPT  0.75
  TFNHOLES 4
```

```
  TFOSPB 20.
  TFOSPT  0.75
  TSLABW 75.
  TSPL 45. 82.
  TSSP 123. 216. 216. 216. 216. 216. 216. 216. 216.
  WCONC 150.
GO
```

```
WAITSFIELD
Girder 2 : Input File : Definition
Mon Dec 21 09:06:32 2015


ID: WAITSFIELD
CONDITIONS
  CLASS B SURFACE
  ENGLISH INPUT
  ENGLISH OUTPUT
  FINE RESOLUTION OUTPUT
  FLANGE BOLTS NOT STAGGERED
  FULL DEPTH CONNECTION PLATES
  INCLUDE SLAB FILLER IN SECTION PROPERTIES
  INTERMEDIATE TRANSVERSE STIFFENERS ONE SIDE OF WEB
  LRFD METHOD
  M270-50W BOTTOM FLANGE STEEL
  M270-50W STIFFENER STEEL
  M270-50W TOP FLANGE STEEL
  M270-50W WEB STEEL
  RATE MODE
  SINGLE BEARING STIFFENERS EACH SIDE
  SYSTEM FORCES
DATA
  ADTT 245
  BFISPB 10.5
  BFISPT 1.
  BFNHOLES 6
  BFOSPB 24.
  BFOSPT 1.
  BR 10.25 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5
  10.5 7.5 10.5 7.5 10.25
  BSPL 45. 82.
  EDGEH 1.5
  EDGEV 1.5
  EDGEW 2.125
  ESLABW 90.
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FBLTDIAM  0.875
  FILLET 1.
  FPC 4.
  FPEDGE 1.5
  FPEND 1.5
  FSPBSP 3.
  GAGEBF 3.75
  GAGEBFW 3.
  GAGEH 3.
  GAGETF 5.75
  GAGETFW 2.75
  GAGEV 3.5
  IGIRD 2
  LIFE 100
  NBLTB 48
  NBLTT 32
  NLINEH 15
  NLINEV 4
  NSTUDL 2
  SLABT 9.
  SLABWEAR  0.5
  SPL 122.
  SPLBFT 1.25 1.75 1.25
  SPLBFW 24.
  SPLT  0.5
  SPLTFT 1.25 1.5 1.25
  SPLTFW 20.
  SPLTST  0.5
  SPLTSW 9.5
  SPLTYP 1
  SPLWD 60.
  SPLWT  0.75
  SPN 172.
  SS 1.
  STD  0.875
  STFGAP  0.
  STH 7.
  SUPBST 1.
  SUPBSW 9.5
  TFISPB 8.75
  TFISPT  0.75
```

```
  TFNHOLES 4
  TFOSPB 20.
  TFOSPT  0.75
  TSLABW 90.
  TSPL 45. 82.
  TSSP 123. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90.
  126. 90. 126. 90.
  WCONC 150.
GO
```

WAITSFIELD
Girder 3 : Input File : Definition
Mon Dec 21 09:06:38 2015

```
ID: WAITSFIELD
CONDITIONS
  CLASS B SURFACE
  ENGLISH INPUT
  ENGLISH OUTPUT
  FINE RESOLUTION OUTPUT
  FLANGE BOLTS NOT STAGGERED
  FULL DEPTH CONNECTION PLATES
  INCLUDE SLAB FILLER IN SECTION PROPERTIES
  INTERMEDIATE TRANSVERSE STIFFENERS ONE SIDE OF WEB
  LRFD METHOD
  M270-50W BOTTOM FLANGE STEEL
  M270-50W STIFFENER STEEL
  M270-50W TOP FLANGE STEEL
  M270-50W WEB STEEL
  RATE MODE
  SINGLE BEARING STIFFENERS EACH SIDE
  SYSTEM FORCES
DATA
  ADTT 245
  BFISPB 10.5
  BFISPT 1.
  BFNHOLES 6
  BFOSPB 24.
  BFOSPT 1.
  BR 10.25 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5
  10.5 7.5 10.5 7.5 10.25
  BSPL 45. 82.
  EDGEH 1.5
  EDGEV 1.5
  EDGEW 2.125
  ESLABW 90.
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FBLTDIAM  0.875
  FILLET 1.
  FPC 4.
  FPEDGE 1.5
  FPEND 1.5
  FSPBSP 3.
  GAGEBF 3.75
  GAGEBFW 3.
  GAGEH 3.
  GAGETF 5.75
  GAGETFW 2.75
  GAGEV 3.5
  IGIRD 3
  LIFE 100
  NBLTB 48
  NBLTT 32
  NLINEH 15
  NLINEV 4
  NSTUDL 2
  SLABT 9.
  SLABWEAR  0.5
  SPL 122.
  SPLBFT 1.25 1.75 1.25
  SPLBFW 24.
  SPLT  0.5
  SPLTFT 1.25 1.5 1.25
  SPLTFW 20.
  SPLTST  0.5
  SPLTSW 9.5
  SPLTYP 1
  SPLWD 60.
  SPLWT  0.75
  SPN 172.
  SS 1.
  STD  0.875
  STFGAP  0.
  STH 7.
  SUPBST 1.
  SUPBSW 9.5
  TFISPB 8.75
  TFISPT  0.75
```

```
  TFNHOLES 4
  TFOSPB 20.
  TFOSPT  0.75
  TSLABW 90.
  TSPL 45. 82.
  TSSP 123. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90.
  126. 90. 126. 90.
  WCONC 150.
GO
```

WAITSFIELD
Girder 4 : Input File : Definition
Mon Dec 21 09:06:44 2015

```
ID: WAITSFIELD
CONDITIONS
  CLASS B SURFACE
  ENGLISH INPUT
  ENGLISH OUTPUT
  FINE RESOLUTION OUTPUT
  FLANGE BOLTS NOT STAGGERED
  FULL DEPTH CONNECTION PLATES
  INCLUDE SLAB FILLER IN SECTION PROPERTIES
  INTERMEDIATE TRANSVERSE STIFFENERS ONE SIDE OF WEB
  LRFD METHOD
  M270-50W BOTTOM FLANGE STEEL
  M270-50W STIFFENER STEEL
  M270-50W TOP FLANGE STEEL
  M270-50W WEB STEEL
  RATE MODE
  SINGLE BEARING STIFFENERS EACH SIDE
  SYSTEM FORCES
DATA
  ADTT 245
  BFISPB 10.5
  BFISPT 1.
  BFNHOLES 6
  BFOSPB 24.
  BFOSPT 1.
  BR 10.25 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5 10.5 7.5
  10.5 7.5 10.5 7.5 10.25
  BSPL 45. 82.
  EDGEH 1.5
  EDGEV 1.5
  EDGEW 2.125
  ESLABW 90.
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FBLTDIAM  0.875
  FILLET 1.
  FPC 4.
  FPEDGE 1.5
  FPEND 1.5
  FSPBSP 3.
  GAGEBF 3.75
  GAGEBFW 3.
  GAGEH 3.
  GAGETF 5.75
  GAGETFW 2.75
  GAGEV 3.5
  IGIRD 4
  LIFE 100
  NBLTB 48
  NBLTT 32
  NLINEH 15
  NLINEV 4
  NSTUDL 2
  SLABT 9.
  SLABWEAR  0.5
  SPL 122.
  SPLBFT 1.25 1.75 1.25
  SPLBFW 24.
  SPLT  0.5
  SPLTFT 1.25 1.5 1.25
  SPLTFW 20.
  SPLTST  0.5
  SPLTSW 9.5
  SPLTYP 1
  SPLWD 60.
  SPLWT  0.75
  SPN 172.
  SS 1.
  STD  0.875
  STFGAP  0.
  STH 7.
  SUPBST 1.
  SUPBSW 9.5
  TFISPB 8.75
  TFISPT  0.75
```

```
  TFNHOLES 4
  TFOSPB 20.
  TFOSPT  0.75
  TSLABW 90.
  TSPL 45. 82.
  TSSP 123. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90. 126. 90.
  126. 90. 126. 90.
  WCONC 150.
GO
```

```
WAITSFIELD
Girder 5 : Input File : Definition
Mon Dec 21 09:06:55 2015


ID: WAITSFIELD
CONDITIONS
  CLASS B SURFACE
  ENGLISH INPUT
  ENGLISH OUTPUT
  FINE RESOLUTION OUTPUT
  FLANGE BOLTS NOT STAGGERED
  FULL DEPTH CONNECTION PLATES
  INCLUDE SLAB FILLER IN SECTION PROPERTIES
  INTERMEDIATE TRANSVERSE STIFFENERS ONE SIDE OF WEB
  LRFD METHOD
  M270-50W BOTTOM FLANGE STEEL
  M270-50W STIFFENER STEEL
  M270-50W TOP FLANGE STEEL
  M270-50W WEB STEEL
  RATE MODE
  SINGLE BEARING STIFFENERS EACH SIDE
  SYSTEM FORCES
DATA
  ADTT 245
  BFISPB 10.5
  BFISPT 1.
  BFNHOLES 6
  BFOSPB 24.
  BFOSPT 1.
  BR 17.75 18. 18. 18. 18. 18. 18. 18. 18. 10.25
  BSPL 45. 82.
  EDGEH 1.5
  EDGEV 1.5
  EDGEW 2.125
  ESLABW 75.
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FBLTDIAM  0.875
  FILLET 1.
  FPC 4.
  FPEDGE 1.5
  FPEND 1.5
  FSPBSP 3.
  GAGEBF 3.75
  GAGEH 3.
  GAGETF 5.75
  GAGETFW 2.75
  GAGEV 3.5
  IGIRD 5
  LIFE 100
  NBLTB 48
  NBLTT 32
  NLINEH 15
  NLINEV 4
  NSTUDL 2
  SLABT 9.
  SLABWEAR  0.5
  SPL 122.
  SPLBFT 1.25 1.75 1.25
  SPLBFW 24.
  SPLT  0.5
  SPLTFT 1.25 1.5 1.25
  SPLTFW 20.
  SPLTST  0.5
  SPLTSW 9.5
  SPLTYP 1
  SPLWD 60.
  SPLWT  0.75
  SPN 172.
  SS 1.
  STD  0.875
  STFGAP  0.
  STH 7.
  SUPBST 1.
  SUPBSW 9.5
  TFISPB 8.75
  TFISPT  0.75
  TFNHOLES 4
  TFOSPB 20.
```

```
  TFOSPT  0.75
  TSLABW 75.
  TSPL 45. 82.
  TSSP 213. 216. 216. 216. 216. 216. 216. 216. 216.
  WCONC 150.
GO
```

```
WAITSFIELD
Bracing : Input File : Definition
Mon Dec 21 09:07:01 2015


ID: WAITSFIELD
CONDITIONS
  ALL SHAPES L4X4X0.5 FOR GROUP 1
  BOTTOM CHORD L4X4X0.5 LONG LEG CONNECTED FOR GROUP 2
  CATEGORY E FATIGUE AT CONNECTIONS
  DIAGONALS L4X4X0.5 LONG LEG CONNECTED FOR GROUP 2
  ENGLISH INPUT
  ENGLISH OUTPUT
  LRFD METHOD
  M270-50W STEEL
  RATE MODE
  TOP CHORD MC8X22.8 FOR GROUP 2
  TYPE B BRACING FOR GROUP 2
  TYPE C BRACING FOR GROUP 1
DATA
  ADTT 245
  BRL-1 10.6066 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 10.6066
  BRL-2 10.6066 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 10.6066
  BRL-3 10.6066 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 10.6066
  BRL-4 10.6066 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 7.5 10.6066
  ETAD 1.
  ETAI 1.
  ETAR 1.
  GCONNDST 8. 11.
  GRP-1 1 1 1 1 1 1 1 1 1 1 2
  GRP-2 1 1 1 1 1 1 1 1 1 1 2
  GRP-3 1 1 1 1 1 1 1 1 1 1 2
  GRP-4 1 1 1 1 1 1 1 1 1 1 2
  GRPHT 51. 48.
  LIFE 100
GO
```

WAITSFIELD
Girder 1 : Rating Output : Factored Strengths
Mon Dec 21 09:18:21 2015

Girder 1 Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	318.3	928.4 (5)	0.343	100.8 I	11003.0 (20)	0.009
1	17.20	258.5	928.4 (5)	0.278	4924.2 I	13446.8 (20)	0.366
2	34.40	198.5	928.4 (5)	0.214	8712.4 I	13446.8 (20)	0.648
3	51.60	142.6	928.4 (5)	0.154	11359.1 I	15421.9 (20)	0.737
4	68.80	91.2	928.4 (5)	0.098	12869.9 I	15421.9 (20)	0.835
5	86.00	55.0	928.4 (5)	0.059	13267.5 I	15421.9 (20)	0.860
6	103.20	106.3	928.4 (5)	0.115	12617.0 I	15421.9 (20)	0.818
7	120.40	155.6	928.4 (5)	0.168	10914.0 I	15421.9 (20)	0.708
8	137.60	202.6	928.4 (5)	0.218	8201.2 I	13446.8 (20)	0.610
9	154.80	245.2	928.4 (5)	0.264	4528.6 I	13446.8 (20)	0.337
10	172.00	290.6	928.4 (5)	0.313	22.9 I	13446.8 (20)	0.002

Absolute values of factored moment are shown with determining strength

Shear Strength at Intermediate Transverse Stiffeners
not Located at a Tenth Point

10.25	282.7	1174.9 (3)	0.241
28.25	220.0	928.4 (5)	0.237
46.25	160.0	928.4 (5)	0.172
64.25	104.8	928.4 (5)	0.113
82.25	62.9	928.4 (5)	0.068
100.25	97.5	928.4 (5)	0.105
118.25	149.5	928.4 (5)	0.161
136.25	198.9	928.4 (5)	0.214
154.25	243.8	928.4 (5)	0.263

(3) Vp(C+.87(1-C)/sqrt(1+(Do/D)**2))
(5) C 0.58 Fyw D t
(20) Compact for pos. mom, Appendix A criteria for neg. mom

WAITSFIELD
Girder 2 : Rating Output : Factored Strengths
Mon Dec 21 09:18:12 2015

Girder 2 Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	350.6	928.4 (5)	0.378	91.7 I	11224.9 (20)	0.008
1	17.20	258.4	1241.6 (3)	0.208	5000.4 I	14217.6 (20)	0.352
2	34.40	205.1	1241.6 (3)	0.165	8689.9 I	14217.6 (20)	0.611
3	51.60	157.5	1241.6 (3)	0.127	11312.0 I	16800.4 (20)	0.673
4	68.80	109.0	1241.6 (3)	0.088	12871.5 I	16800.4 (20)	0.766
5	86.00	71.2	1241.6 (3)	0.057	13399.2 I	16800.4 (20)	0.798
6	103.20	122.3	1241.6 (3)	0.098	12902.4 I	16800.4 (20)	0.768
7	120.40	170.0	1241.6 (3)	0.137	11346.0 I	16800.4 (20)	0.675
8	137.60	217.0	1241.6 (3)	0.175	8717.5 I	14217.6 (20)	0.613
9	154.80	266.6	1241.6 (3)	0.215	4995.5 I	14217.6 (20)	0.351
10	172.00	341.9	928.4 (5)	0.368	55.3 I	14217.6 (20)	0.004

Absolute values of factored moment are shown with determining strength

Shear Strength at Intermediate Transverse Stiffeners
not Located at a Tenth Point

10.25	295.7	1174.9 (3)	0.252
17.75	256.7	1170.2 (3)	0.219
28.25	224.1	1170.2 (3)	0.192
35.75	201.3	1170.2 (3)	0.172
46.25	172.3	1170.2 (3)	0.147
53.75	151.4	1170.2 (3)	0.129
64.25	121.9	1170.2 (3)	0.104
71.75	102.5	1170.2 (3)	0.088
82.25	79.4	1170.2 (3)	0.068
89.75	82.3	1170.2 (3)	0.070
100.25	113.5	1170.2 (3)	0.097
107.75	134.9	1170.2 (3)	0.115
118.25	164.0	1170.2 (3)	0.140
125.75	184.6	1170.2 (3)	0.158
136.25	213.3	1170.2 (3)	0.182
143.75	234.7	1170.2 (3)	0.201
154.25	265.0	1170.2 (3)	0.226
161.75	297.0	1174.9 (3)	0.253

(3) Vp(C+.87(1-C)/sqrt(1+(Do/D)**2))
(5) C 0.58 Fyw D t
(20) Compact for pos. mom, Appendix A criteria for neg. mom

WAITSFIELD
Girder 3 : Rating Output : Factored Strengths
Mon Dec 21 09:18:05 2015

Girder 3 Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	337.8	928.4 (5)	0.364	59.1 I	11224.9 (20)	0.005
1	17.20	250.3	1241.6 (3)	0.202	4848.6 I	14217.6 (20)	0.341
2	34.40	199.5	1241.6 (3)	0.161	8409.3 I	14217.6 (20)	0.591
3	51.60	153.3	1241.6 (3)	0.123	10882.1 I	16800.4 (20)	0.648
4	68.80	107.9	1241.6 (3)	0.087	12312.5 I	16800.4 (20)	0.733
5	86.00	66.8	1241.6 (3)	0.054	12788.5 I	16800.4 (20)	0.761
6	103.20	115.8	1241.6 (3)	0.093	12319.2 I	16800.4 (20)	0.733
7	120.40	161.2	1241.6 (3)	0.130	10895.1 I	16800.4 (20)	0.649
8	137.60	207.5	1241.6 (3)	0.167	8428.6 I	14217.6 (20)	0.593
9	154.80	258.4	1241.6 (3)	0.208	4874.5 I	14217.6 (20)	0.343
10	172.00	337.6	928.4 (5)	0.364	59.7 I	11224.9 (20)	0.005

Absolute values of factored moment are shown with determining strength

Shear Strength at Intermediate Transverse Stiffeners
not Located at a Tenth Point

10.25	285.6	1174.9 (3)	0.243
17.75	248.7	1170.2 (3)	0.212
28.25	217.7	1170.2 (3)	0.186
35.75	195.9	1170.2 (3)	0.167
46.25	167.7	1170.2 (3)	0.143
53.75	147.6	1170.2 (3)	0.126
64.25	119.9	1170.2 (3)	0.102
71.75	100.9	1170.2 (3)	0.086
82.25	75.8	1170.2 (3)	0.065
89.75	77.5	1170.2 (3)	0.066
100.25	107.4	1170.2 (3)	0.092
107.75	127.8	1170.2 (3)	0.109
118.25	155.5	1170.2 (3)	0.133
125.75	175.6	1170.2 (3)	0.150
136.25	203.9	1170.2 (3)	0.174
143.75	225.7	1170.2 (3)	0.193
154.25	256.7	1170.2 (3)	0.219
161.75	290.4	1174.9 (3)	0.247

(3) Vp(C+.87(1-C)/sqrt(1+(Do/D)**2))
(5) C 0.58 Fyw D t
(20) Compact for pos. mom, Appendix A criteria for neg. mom

WAITSFIELD
Girder 4 : Rating Output : Factored Strengths
Mon Dec 21 09:17:58 2015

Girder 4 Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	341.9	928.4 (5)	0.368	54.7 I	14217.6 (20)	0.004
1	17.20	259.2	1241.6 (3)	0.209	4969.2 I	14217.6 (20)	0.350
2	34.40	209.0	1241.6 (3)	0.168	8697.4 I	14217.6 (20)	0.612
3	51.60	162.0	1241.6 (3)	0.131	11331.9 I	16800.4 (20)	0.675
4	68.80	114.2	1241.6 (3)	0.092	12894.6 I	16800.4 (20)	0.768
5	86.00	67.0	1241.6 (3)	0.054	13397.8 I	16800.4 (20)	0.797
6	103.20	117.1	1241.6 (3)	0.094	12876.5 I	16800.4 (20)	0.766
7	120.40	165.4	1241.6 (3)	0.133	11323.3 I	16800.4 (20)	0.674
8	137.60	213.1	1241.6 (3)	0.172	8707.6 I	14217.6 (20)	0.612
9	154.80	264.5	1241.6 (3)	0.213	5024.3 I	14217.6 (20)	0.353
10	172.00	350.2	928.4 (5)	0.377	92.4 I	11224.9 (20)	0.008

Absolute values of factored moment are shown with determining strength

Shear Strength at Intermediate Transverse Stiffeners
not Located at a Tenth Point

10.25	292.6	1174.9 (3)	0.249
17.75	257.6	1170.2 (3)	0.220
28.25	226.9	1170.2 (3)	0.194
35.75	205.3	1170.2 (3)	0.175
46.25	176.6	1170.2 (3)	0.151
53.75	156.1	1170.2 (3)	0.133
64.25	126.9	1170.2 (3)	0.108
71.75	106.1	1170.2 (3)	0.091
82.25	77.3	1170.2 (3)	0.066
89.75	77.9	1170.2 (3)	0.067
100.25	108.5	1170.2 (3)	0.093
107.75	129.9	1170.2 (3)	0.111
118.25	159.4	1170.2 (3)	0.136
125.75	180.3	1170.2 (3)	0.154
136.25	209.4	1170.2 (3)	0.179
143.75	231.5	1170.2 (3)	0.198
154.25	262.9	1170.2 (3)	0.225
161.75	299.1	1174.9 (3)	0.255

(3) Vp(C+.87(1-C)/sqrt(1+(Do/D)**2))
(5) C 0.58 Fyw D t
(20) Compact for pos. mom, Appendix A criteria for neg. mom

WAITSFIELD
Girder 5 : Rating Output : Factored Strengths
Mon Dec 21 09:17:19 2015

Girder 5 Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	290.5	928.4 (5)	0.313	22.7 I	13446.8 (20)	0.002
1	17.20	235.5	928.4 (5)	0.254	4493.4 I	13446.8 (20)	0.334
2	34.40	192.3	928.4 (5)	0.207	8166.9 I	13446.8 (20)	0.607
3	51.60	144.4	928.4 (5)	0.156	10880.9 I	15421.9 (20)	0.706
4	68.80	94.5	928.4 (5)	0.102	12585.6 I	15421.9 (20)	0.816
5	86.00	50.3	928.4 (5)	0.054	13267.7 I	15421.9 (20)	0.860
6	103.20	90.6	928.4 (5)	0.098	12869.9 I	15421.9 (20)	0.835
7	120.40	142.6	928.4 (5)	0.154	11358.6 I	15421.9 (20)	0.737
8	137.60	199.1	928.4 (5)	0.214	8711.3 I	13446.8 (20)	0.648
9	154.80	262.9	928.4 (5)	0.283	4922.8 I	13446.8 (20)	0.366
10	172.00	317.8	928.4 (5)	0.342	102.6 I	11003.0 (20)	0.009

Absolute values of factored moment are shown with determining strength

Shear Strength at Intermediate Transverse Stiffeners
not Located at a Tenth Point

17.75	234.2	928.4 (5)	0.252
35.75	188.5	928.4 (5)	0.203
53.75	138.2	928.4 (5)	0.149
71.75	86.9	928.4 (5)	0.094
89.75	59.1	928.4 (5)	0.064
107.75	104.4	928.4 (5)	0.112
125.75	160.2	928.4 (5)	0.173
143.75	221.9	928.4 (5)	0.239
161.75	285.1	1174.9 (3)	0.243

(3) Vp(C+.87(1-C)/sqrt(1+(Do/D)**2))
(5) C 0.58 Fyw D t
(20) Compact for pos. mom, Appendix A criteria for neg. mom

WAITSFIELD
Girder 1 : Rating Output : Service Moment
Mon Dec 21 09:20:15 2015

Girder 1 Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	0.	-1.	-1.	22.	-56.	16.	-35.	32.
	8.6	847.	63.	160.	708.	-21.	431.	-13.	288.
1	17.2	1608.	119.	304.	1319.	0.	804.	0.	517.
	25.8	2285.	168.	431.	1856.	0.	1139.	0.	721.
2	34.4	2874.	211.	540.	2312.	0.	1424.	0.	891.
	43.0	3376.	247.	632.	2687.	0.	1658.	0.	1027.
Top flg spl		3482.	254.	650.	2761.	0.	1704.	0.	1054.
Bot flg spl		3482.	254.	650.	2761.	0.	1704.	0.	1054.
3	51.6	3791.	276.	706.	2980.	0.	1841.	0.	1129.
	60.2	4113.	299.	765.	3197.	0.	1978.	0.	1197.
4	68.8	4344.	315.	806.	3335.	0.	2067.	0.	1232.
	77.4	4482.	325.	831.	3397.	0.	2109.	0.	1240.
5	86.0	4527.	328.	838.	3395.	0.	2109.	0.	1234.
	94.6	4478.	325.	829.	3330.	0.	2065.	0.	1222.
6	103.2	4338.	314.	804.	3198.	0.	1981.	0.	1183.
	111.8	4106.	298.	761.	2997.	0.	1856.	0.	1120.
7	120.4	3784.	275.	702.	2735.	0.	1693.	0.	1028.
Top flg spl		3475.	253.	645.	2494.	0.	1541.	0.	940.
Bot flg spl		3475.	253.	645.	2494.	0.	1541.	0.	940.
	129.0	3369.	245.	626.	2412.	0.	1488.	0.	906.
8	137.6	2867.	209.	534.	2032.	0.	1250.	0.	764.
	146.2	2278.	166.	425.	1587.	1.	972.	1.	604.
9	154.8	1604.	117.	299.	1102.	0.	673.	0.	425.
	163.4	845.	62.	158.	578.	-3.	352.	-2.	226.
10	172.0	-1.	0.	0.	14.	-9.	8.	-5.	9.

Bracing

10.2	1000.	74.	189.	831.	-16.	506.	-10.
28.2	2461.	181.	463.	1995.	0.	1226.	0.
46.2	3545.	259.	662.	2806.	0.	1732.	0.
64.2	4233.	308.	786.	3270.	0.	2025.	0.
82.2	4519.	328.	837.	3404.	0.	2114.	0.
100.2	4396.	319.	814.	3251.	0.	2014.	0.
118.2	3873.	281.	719.	2806.	0.	1738.	0.
136.2	2952.	215.	549.	2095.	0.	1289.	0.
154.2	1650.	120.	308.	1134.	0.	693.	0.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	9.	-23.	0.	0.
1	0.	0.	517.	0.	0.	0.
2	0.	0.	891.	0.	0.	0.
Top flg spl	0.	0.	1054.	0.	0.	0.
Bot flg spl	0.	0.	1054.	0.	0.	0.
3	0.	0.	1129.	0.	0.	0.
4	0.	0.	1232.	0.	0.	0.
5	0.	0.	1234.	0.	0.	0.
6	0.	0.	1183.	0.	0.	0.
7	0.	0.	1028.	0.	0.	0.
Top flg spl	0.	0.	940.	0.	0.	0.
Bot flg spl	0.	0.	940.	0.	0.	0.
8	0.	0.	764.	0.	0.	0.
9	0.	0.	425.	0.	0.	0.
10	0.	0.	5.	-3.	0.	0.

WAITSFIELD
Girder 2 : Rating Output : Service Moment
Mon Dec 21 09:20:07 2015

Girder 2 Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	0.	0.	-1.	16.	-51.	14.	-33.	27.
	8.6	897.	63.	161.	705.	-19.	428.	-12.	215.
1	17.2	1705.	120.	306.	1292.	0.	785.	0.	379.
	25.8	2427.	170.	435.	1760.	0.	1077.	0.	516.
2	34.4	3053.	214.	547.	2163.	0.	1327.	0.	636.
	43.0	3584.	251.	642.	2501.	0.	1538.	0.	741.
Top flg spl		3695.	259.	662.	2569.	0.	1579.	0.	765.
Bot flg spl		3695.	259.	662.	2569.	0.	1579.	0.	765.
3	51.6	4020.	282.	720.	2774.	0.	1707.	0.	830.
	60.2	4357.	306.	781.	2986.	0.	1839.	0.	899.
4	68.8	4596.	322.	824.	3136.	0.	1934.	0.	949.
	77.4	4743.	332.	849.	3226.	0.	1993.	0.	972.
5	86.0	4790.	336.	858.	3260.	0.	2015.	0.	986.
	94.6	4736.	332.	849.	3242.	0.	2000.	0.	993.
6	103.2	4587.	322.	823.	3161.	0.	1947.	0.	981.
	111.8	4344.	305.	780.	3015.	0.	1856.	0.	953.
7	120.4	4006.	282.	720.	2803.	0.	1724.	0.	902.
Top flg spl		3682.	259.	663.	2596.	0.	1596.	0.	848.
Bot flg spl		3682.	259.	663.	2596.	0.	1596.	0.	848.
	129.0	3572.	252.	643.	2528.	0.	1552.	0.	831.
8	137.6	3044.	215.	549.	2183.	0.	1339.	0.	733.
	146.2	2423.	171.	438.	1777.	3.	1092.	2.	615.
9	154.8	1709.	121.	309.	1282.	0.	790.	0.	456.
	163.4	901.	64.	163.	701.	-8.	433.	-5.	259.
10	172.0	-1.	0.	0.	32.	-21.	20.	-13.	22.

Bracing

10.2	1059.	74.	190.	825.	-15.	501.	-9.
17.8	1753.	123.	314.	1326.	1.	806.	1.
28.2	2615.	183.	468.	1882.	0.	1152.	0.
35.8	3143.	220.	563.	2221.	0.	1363.	0.
46.2	3761.	264.	674.	2611.	0.	1605.	0.
53.8	4113.	288.	737.	2833.	0.	1744.	0.
64.2	4481.	314.	803.	3063.	0.	1888.	0.
71.8	4656.	327.	835.	3174.	0.	1958.	0.
82.2	4782.	335.	856.	3252.	0.	2010.	0.
89.8	4780.	335.	856.	3257.	0.	2013.	0.
100.2	4648.	326.	834.	3196.	0.	1969.	0.
107.8	4470.	314.	803.	3091.	0.	1903.	0.
118.2	4099.	288.	737.	2862.	0.	1761.	0.
125.8	3747.	264.	674.	2639.	0.	1622.	0.
136.2	3133.	221.	565.	2242.	0.	1375.	0.
143.8	2610.	184.	471.	1893.	0.	1162.	0.
154.2	1758.	124.	318.	1317.	0.	811.	0.
161.8	1063.	75.	193.	819.	-6.	505.	-4.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	7.	-20.	0.	0.
1	0.	0.	379.	0.	0.	0.
2	0.	0.	636.	0.	0.	0.
Top flg spl	0.	0.	765.	0.	0.	0.
Bot flg spl	0.	0.	765.	0.	0.	0.
3	0.	0.	830.	0.	0.	0.
4	0.	0.	949.	0.	0.	0.
5	0.	0.	986.	0.	0.	0.

6	0.	0.	981.	0.	0.	0.
7	0.	0.	902.	0.	0.	0.
Top flg spl	0.	0.	848.	0.	0.	0.
Bot flg spl	0.	0.	848.	0.	0.	0.
8	0.	0.	733.	0.	0.	0.
9	0.	0.	456.	0.	0.	0.
10	0.	0.	13.	-9.	0.	0.

WAITSFIELD
Girder 3 : Rating Output : Service Moment
Mon Dec 21 09:20:00 2015

Girder 3 Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	-1.	0.	0.	17.	-33.	11.	-21.	20.
	8.6	903.	63.	161.	655.	-12.	393.	-8.	199.
1	17.2	1718.	120.	306.	1196.	0.	721.	0.	347.
	25.8	2448.	169.	433.	1625.	0.	985.	0.	462.
2	34.4	3083.	213.	543.	1985.	0.	1207.	0.	552.
	43.0	3623.	249.	637.	2278.	0.	1389.	0.	619.
Top flg spl		3735.	257.	656.	2333.	0.	1422.	0.	628.
Bot flg spl		3735.	257.	656.	2333.	0.	1422.	0.	628.
3	51.6	4066.	279.	713.	2503.	0.	1529.	0.	662.
	60.2	4411.	302.	772.	2672.	0.	1636.	0.	691.
4	68.8	4658.	319.	815.	2782.	0.	1708.	0.	703.
	77.4	4812.	329.	840.	2847.	0.	1752.	0.	704.
5	86.0	4864.	332.	849.	2869.	0.	1767.	0.	704.
	94.6	4812.	329.	840.	2849.	0.	1752.	0.	709.
6	103.2	4664.	319.	815.	2782.	0.	1708.	0.	704.
	111.8	4420.	302.	772.	2672.	0.	1636.	0.	691.
7	120.4	4077.	279.	713.	2503.	0.	1529.	0.	662.
Top flg spl		3748.	257.	656.	2332.	0.	1422.	0.	628.
Bot flg spl		3748.	257.	656.	2332.	0.	1422.	0.	628.
	129.0	3636.	249.	637.	2278.	0.	1388.	0.	619.
8	137.6	3099.	213.	543.	1985.	0.	1207.	0.	552.
	146.2	2467.	169.	433.	1639.	4.	991.	3.	465.
9	154.8	1740.	120.	306.	1195.	0.	720.	0.	347.
	163.4	917.	63.	161.	655.	-13.	393.	-8.	199.
10	172.0	-1.	0.	0.	17.	-33.	11.	-21.	20.

Bracing

10.2	1066.	74.	190.	767.	-9.	461.	-6.
17.8	1767.	123.	314.	1227.	1.	740.	0.
28.2	2639.	182.	466.	1734.	0.	1052.	0.
35.8	3174.	219.	559.	2036.	0.	1239.	0.
46.2	3803.	261.	668.	2368.	0.	1444.	0.
53.8	4162.	285.	730.	2551.	0.	1559.	0.
64.2	4539.	311.	795.	2727.	0.	1672.	0.
71.8	4721.	323.	826.	2810.	0.	1727.	0.
82.2	4854.	332.	847.	2864.	0.	1764.	0.
89.8	4855.	332.	847.	2864.	0.	1764.	0.
100.2	4726.	323.	826.	2810.	0.	1727.	0.
107.8	4546.	311.	795.	2727.	0.	1672.	0.
118.2	4172.	285.	730.	2551.	0.	1559.	0.
125.8	3815.	261.	668.	2367.	0.	1444.	0.
136.2	3190.	219.	559.	2035.	0.	1238.	0.
143.8	2657.	182.	466.	1734.	0.	1052.	0.
154.2	1789.	123.	314.	1227.	1.	739.	0.
161.8	1082.	74.	190.	766.	-9.	460.	-6.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	7.	-13.	0.	0.
1	0.	0.	347.	0.	0.	0.
2	0.	0.	552.	0.	0.	0.
Top flg spl	0.	0.	628.	0.	0.	0.
Bot flg spl	0.	0.	628.	0.	0.	0.
3	0.	0.	662.	0.	0.	0.
4	0.	0.	703.	0.	0.	0.
5	0.	0.	704.	0.	0.	0.

6	0.	0.	704.	0.	0.	0.
7	0.	0.	662.	0.	0.	0.
Top flg spl	0.	0.	628.	0.	0.	0.
Bot flg spl	0.	0.	628.	0.	0.	0.
8	0.	0.	552.	0.	0.	0.
9	0.	0.	347.	0.	0.	0.
10	0.	0.	7.	-13.	0.	0.

WAITSFIELD
Girder 4 : Rating Output : Service Moment
Mon Dec 21 09:19:53 2015

Girder 4 Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	-1.	0.	0.	32.	-21.	20.	-13.	21.
	8.6	887.	64.	163.	701.	-8.	433.	-5.	259.
1	17.2	1687.	121.	309.	1283.	0.	790.	0.	457.
	25.8	2404.	171.	437.	1769.	0.	1085.	0.	609.
2	34.4	3027.	215.	549.	2184.	0.	1339.	0.	733.
	43.0	3557.	252.	643.	2529.	0.	1552.	0.	831.
Top flg spl		3668.	259.	663.	2597.	0.	1596.	0.	848.
Bot flg spl		3668.	259.	663.	2597.	0.	1596.	0.	848.
3	51.6	3994.	282.	720.	2804.	0.	1724.	0.	902.
	60.2	4335.	305.	780.	3015.	0.	1856.	0.	953.
4	68.8	4581.	322.	823.	3161.	0.	1947.	0.	981.
	77.4	4735.	332.	849.	3238.	0.	2000.	0.	989.
5	86.0	4790.	336.	858.	3260.	0.	2015.	0.	986.
	94.6	4743.	332.	849.	3228.	0.	1993.	0.	977.
6	103.2	4600.	322.	824.	3136.	0.	1934.	0.	949.
	111.8	4363.	305.	781.	2986.	0.	1839.	0.	899.
7	120.4	4029.	282.	720.	2774.	0.	1707.	0.	830.
Top flg spl		3707.	259.	662.	2569.	0.	1579.	0.	764.
Bot flg spl		3707.	259.	662.	2569.	0.	1579.	0.	764.
	129.0	3597.	251.	642.	2501.	0.	1537.	0.	741.
8	137.6	3068.	214.	547.	2163.	0.	1327.	0.	636.
	146.2	2444.	170.	434.	1778.	6.	1085.	4.	519.
9	154.8	1725.	120.	306.	1292.	0.	785.	0.	378.
	163.4	910.	63.	161.	704.	-19.	426.	-12.	215.
10	172.0	0.	0.	-1.	16.	-52.	9.	-33.	28.

Bracing

10.2	1047.	75.	193.	820.	-6.	506.	-4.
17.8	1735.	124.	318.	1317.	0.	811.	0.
28.2	2591.	184.	471.	1894.	0.	1162.	0.
35.8	3117.	221.	565.	2242.	0.	1376.	0.
46.2	3734.	264.	674.	2639.	0.	1622.	0.
53.8	4088.	288.	737.	2863.	0.	1761.	0.
64.2	4462.	314.	803.	3091.	0.	1903.	0.
71.8	4643.	326.	834.	3196.	0.	1969.	0.
82.2	4778.	335.	856.	3257.	0.	2013.	0.
89.8	4782.	335.	856.	3252.	0.	2010.	0.
100.2	4660.	327.	835.	3174.	0.	1958.	0.
107.8	4486.	314.	803.	3063.	0.	1888.	0.
118.2	4122.	288.	737.	2833.	0.	1744.	0.
125.8	3772.	264.	674.	2611.	0.	1605.	0.
136.2	3157.	220.	563.	2220.	0.	1363.	0.
143.8	2632.	183.	468.	1881.	0.	1152.	0.
154.2	1773.	123.	315.	1326.	1.	806.	1.
161.8	1073.	74.	190.	825.	-15.	499.	-9.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	13.	-8.	0.	0.
1	0.	0.	457.	0.	0.	0.
2	0.	0.	733.	0.	0.	0.
Top flg spl	0.	0.	848.	0.	0.	0.
Bot flg spl	0.	0.	848.	0.	0.	0.
3	0.	0.	902.	0.	0.	0.
4	0.	0.	981.	0.	0.	0.
5	0.	0.	986.	0.	0.	0.

6	0.	0.	949.	0.	0.	0.
7	0.	0.	830.	0.	0.	0.
Top flg spl	0.	0.	764.	0.	0.	0.
Bot flg spl	0.	0.	764.	0.	0.	0.
8	0.	0.	636.	0.	0.	0.
9	0.	0.	378.	0.	0.	0.
10	0.	0.	7.	-20.	0.	0.

WAITSFIELD
Girder 5 : Rating Output : Service Moment
Mon Dec 21 09:19:38 2015

Girder 5 Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	-1.	0.	0.	13.	-8.	8.	-5.	9.
	8.6	826.	62.	158.	578.	-3.	352.	-2.	227.
1	17.2	1575.	117.	299.	1102.	0.	673.	0.	425.
	25.8	2250.	166.	425.	1596.	0.	978.	0.	604.
2	34.4	2839.	209.	534.	2032.	0.	1250.	0.	764.
	43.0	3341.	245.	626.	2412.	0.	1489.	0.	906.
Top flg spl		3448.	253.	646.	2494.	0.	1542.	0.	940.
Bot flg spl		3448.	253.	646.	2494.	0.	1542.	0.	940.
3	51.6	3757.	275.	702.	2736.	0.	1693.	0.	1029.
	60.2	4080.	298.	761.	2997.	0.	1857.	0.	1120.
4	68.8	4312.	315.	804.	3198.	0.	1981.	0.	1184.
	77.4	4469.	325.	829.	3328.	0.	2066.	0.	1216.
5	86.0	4527.	328.	838.	3395.	0.	2109.	0.	1234.
	94.6	4482.	325.	831.	3402.	0.	2111.	0.	1245.
6	103.2	4344.	315.	806.	3335.	0.	2067.	0.	1232.
	111.8	4113.	299.	765.	3197.	0.	1978.	0.	1197.
7	120.4	3791.	276.	706.	2980.	0.	1841.	0.	1129.
Top flg spl		3482.	254.	650.	2760.	0.	1703.	0.	1053.
Bot flg spl		3482.	254.	650.	2760.	0.	1703.	0.	1053.
	129.0	3376.	247.	632.	2686.	0.	1658.	0.	1027.
8	137.6	2874.	211.	540.	2312.	0.	1424.	0.	891.
	146.2	2284.	169.	431.	1853.	7.	1135.	4.	717.
9	154.8	1608.	119.	304.	1318.	0.	804.	0.	517.
	163.4	847.	63.	160.	708.	-21.	430.	-13.	288.
10	172.0	0.	-1.	-2.	22.	-57.	13.	-36.	33.

Bracing

	17.8	1620.	120.	308.	1135.	0.	693.	0.
	35.8	2923.	215.	549.	2096.	0.	1290.	0.
	53.8	3846.	281.	719.	2807.	0.	1738.	0.
	71.8	4373.	319.	814.	3251.	0.	2014.	0.
	89.8	4521.	328.	837.	3404.	0.	2114.	0.
	107.8	4233.	308.	786.	3270.	0.	2025.	0.
	125.8	3545.	259.	662.	2805.	0.	1732.	0.
	143.8	2461.	181.	463.	1994.	0.	1225.	0.
	161.8	1000.	74.	189.	831.	-16.	505.	-10.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	5.	-3.	0.	0.
1	0.	0.	425.	0.	0.	0.
2	0.	0.	764.	0.	0.	0.
Top flg spl	0.	0.	940.	0.	0.	0.
Bot flg spl	0.	0.	940.	0.	0.	0.
3	0.	0.	1029.	0.	0.	0.
4	0.	0.	1184.	0.	0.	0.
5	0.	0.	1234.	0.	0.	0.
6	0.	0.	1232.	0.	0.	0.
7	0.	0.	1129.	0.	0.	0.
Top flg spl	0.	0.	1053.	0.	0.	0.
Bot flg spl	0.	0.	1053.	0.	0.	0.
8	0.	0.	891.	0.	0.	0.
9	0.	0.	517.	0.	0.	0.
10	0.	0.	10.	-23.	0.	0.

WAITSFIELD
Girder 1 : Rating Output : Service Shear
Mon Dec 21 09:20:26 2015

Girder 1 Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	103.41	7.77	19.85	85.47	0.00	52.09	34.66
	8.60	93.49	6.95	17.75	77.87	0.34	46.95	31.45
1	17.20	83.53	6.14	15.68	70.23	0.00	41.90	28.68
	25.80	73.54	5.34	13.65	62.19	-1.03	36.76	26.36
2	34.40	63.50	4.56	11.65	54.84	-2.76	32.11	24.48
	43.00	53.18	3.79	9.68	48.18	-7.81	27.95	24.25
Top flg spl		50.73	3.62	9.26	46.70	-10.99	27.10	25.00
Bot flg spl		50.73	3.62	9.26	46.70	-10.99	27.10	25.00
3	51.60	42.65	3.04	7.76	42.21	-15.30	24.27	25.29
	60.20	31.99	2.27	5.81	37.05	-19.97	20.93	25.61
4	68.80	21.17	1.50	3.85	32.65	-23.57	17.97	25.89
	77.40	10.69	0.74	0.99	28.90	-27.30	16.66	26.12
5	86.00	-0.37	-0.02	-0.05	25.39	-31.11	-16.92	26.24
	94.60	-12.01	-0.78	-1.99	22.20	-35.02	-19.48	26.24
6	103.20	-24.22	-1.54	-3.93	19.07	-39.01	-22.16	26.12
	111.80	-34.68	-2.30	-5.88	15.86	-43.56	-25.08	25.88
7	120.40	-44.92	-3.06	-7.82	12.98	-47.96	-27.92	25.64
Top flg spl		-52.63	-3.64	-9.31	11.01	-51.22	-30.05	25.44
Bot flg spl		-52.63	-3.64	-9.31	11.01	-51.22	-30.05	25.44
	129.00	-54.90	-3.82	-9.76	10.59	-52.35	-30.78	25.29
8	137.60	-64.73	-4.57	-11.67	8.24	-56.26	-33.36	25.13
	146.20	-74.42	-5.30	-13.52	5.88	-59.68	-35.32	25.17
9	154.80	-83.94	-6.03	-15.42	3.66	-62.62	-37.64	25.41
	163.40	-93.52	-6.80	-17.37	1.58	-66.33	-40.33	26.30
10	172.00	-103.09	-7.58	-19.37	-0.37	-70.40	-43.37	27.70

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	34.66	0.00	0.00	0.00
1	28.68	0.00	0.00	0.00
2	23.01	0.00	0.00	0.00
Top flg spl	20.28	0.00	0.00	0.00
Bot flg spl	20.28	0.00	0.00	0.00
3	18.70	0.00	0.00	0.00
4	15.02	0.00	0.00	0.00
5	-14.39	0.00	0.00	0.00
6	-17.37	0.00	0.00	0.00
7	-20.07	0.00	0.00	0.00
Top flg spl	-21.00	0.00	0.00	0.00
Bot flg spl	-21.00	0.00	0.00	0.00
8	-22.36	0.00	0.00	0.00
9	-24.23	0.00	0.00	0.00
10	-27.97	0.00	0.00	0.00

WAITSFIELD
Girder 2 : Rating Output : Service Shear
Mon Dec 21 09:20:36 2015

Girder 2 Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	111.49	7.76	19.84	98.16	0.18	59.18	32.76
	8.60	96.30	6.96	17.79	82.23	1.32	49.11	26.55
1	17.20	82.89	6.18	15.80	70.50	0.13	41.70	23.34
	25.80	71.26	5.43	13.92	64.61	-3.39	38.01	23.11
2	34.40	61.41	4.70	12.01	59.66	-9.24	34.87	25.87
	43.00	50.52	3.94	10.06	55.66	-17.29	32.28	27.72
Top flg spl		47.93	3.76	9.60	55.35	-21.91	32.02	28.86
Bot flg spl		47.93	3.76	9.60	55.35	-21.91	32.02	28.86
3	51.60	39.52	3.16	8.07	52.60	-27.59	30.24	29.96
	60.20	28.55	2.37	6.05	48.94	-32.42	27.91	30.51
4	68.80	17.58	1.56	4.00	45.20	-34.36	25.56	30.07
	77.40	9.25	0.78	1.01	41.56	-37.12	22.96	29.89
5	86.00	-0.73	0.00	-0.01	38.25	-40.15	-22.24	29.76
	94.60	-12.36	-0.77	-1.98	35.34	-43.46	-24.27	29.69
6	103.20	-25.66	-1.55	-3.96	32.60	-47.04	-26.43	29.68
	111.80	-36.38	-2.33	-5.96	30.28	-50.79	-28.74	29.74
7	120.40	-47.02	-3.11	-7.95	27.61	-54.51	-31.10	29.52
Top flg spl		-55.13	-3.71	-9.47	25.31	-57.34	-32.94	29.14
Bot flg spl		-55.13	-3.71	-9.47	25.31	-57.34	-32.94	29.14
	129.00	-57.53	-3.88	-9.93	25.29	-57.99	-33.35	28.70
8	137.60	-68.04	-4.66	-11.90	21.18	-61.86	-35.94	28.19
	146.20	-78.53	-5.43	-13.86	13.66	-66.11	-37.89	27.97
9	154.80	-89.01	-6.21	-15.86	7.58	-70.74	-42.11	28.04
	163.40	-99.63	-7.01	-17.92	2.94	-80.49	-48.57	30.78
10	172.00	-110.33	-7.84	-20.03	-0.25	-93.78	-57.30	35.40

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	32.90	0.00	0.00	0.00
1	23.43	0.00	0.00	0.00
2	21.41	0.00	0.00	0.00
Top flg spl	20.96	0.00	0.00	0.00
Bot flg spl	20.96	0.00	0.00	0.00
3	20.50	0.00	0.00	0.00
4	18.59	0.00	0.00	0.00
5	16.54	0.00	0.00	0.00
6	-15.35	0.00	0.00	0.00
7	-17.51	0.00	0.00	0.00
Top flg spl	-18.34	0.00	0.00	0.00
Bot flg spl	-18.34	0.00	0.00	0.00
8	-19.65	0.00	0.00	0.00
9	-23.91	0.00	0.00	0.00
10	-35.58	0.00	0.00	0.00

WAITSFIELD
Girder 3 : Rating Output : Service Shear
Mon Dec 21 09:20:59 2015

WAITSFIELD
Girder 4 : Rating Output : Service Shear
Mon Dec 21 09:21:07 2015

Girder 4 Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	110.32	7.84	20.02	93.83	0.25	57.31	35.42
	8.60	95.37	7.01	17.92	80.95	1.19	48.88	29.52
1	17.20	82.20	6.21	15.86	71.37	0.02	42.54	25.75
	25.80	70.82	5.42	13.88	66.53	-3.26	39.18	24.13
2	34.40	61.23	4.66	11.90	62.16	-8.65	36.20	24.65
	43.00	50.73	3.88	9.93	58.26	-16.78	33.60	26.20
Top flg spl		48.32	3.71	9.47	57.66	-21.84	33.22	27.90
Bot flg spl		48.32	3.71	9.47	57.66	-21.84	33.22	27.90
3	51.60	40.21	3.11	7.95	54.83	-27.42	31.39	29.14
	60.20	29.57	2.33	5.96	51.09	-31.68	29.02	29.87
4	68.80	18.85	1.55	3.96	47.31	-32.51	26.68	29.59
	77.40	10.65	0.77	0.99	40.80	-34.91	22.85	28.21
5	86.00	0.71	0.00	0.01	36.49	-37.79	-21.36	27.80
	94.60	-10.97	-0.78	-1.99	35.99	-41.15	-23.30	28.36
6	103.20	-24.39	-1.56	-4.00	34.41	-44.98	-25.39	29.88
	111.80	-35.36	-2.37	-6.05	31.34	-48.64	-27.72	30.12
7	120.40	-46.33	-3.16	-8.07	28.03	-52.27	-30.04	30.06
Top flg spl		-54.74	-3.76	-9.60	25.33	-55.03	-31.81	29.81
Bot flg spl		-54.74	-3.76	-9.60	25.33	-55.03	-31.81	29.81
	129.00	-57.33	-3.94	-10.06	24.83	-55.52	-32.06	29.67
8	137.60	-68.22	-4.70	-12.01	20.70	-59.41	-34.65	29.04
	146.20	-79.01	-5.45	-13.87	15.03	-63.94	-36.74	28.16
9	154.80	-89.70	-6.18	-15.80	9.67	-69.11	-41.51	27.05
	163.40	-100.48	-6.96	-17.79	4.59	-81.18	-48.97	28.94
10	172.00	-111.30	-7.76	-19.84	-0.19	-98.06	-59.11	32.75

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	35.60	0.00	0.00	0.00
1	25.77	0.00	0.00	0.00
2	20.02	0.00	0.00	0.00
Top flg spl	18.64	0.00	0.00	0.00
Bot flg spl	18.64	0.00	0.00	0.00
3	17.79	0.00	0.00	0.00
4	15.59	0.00	0.00	0.00
5	-16.39	0.00	0.00	0.00
6	-18.32	0.00	0.00	0.00
7	-20.29	0.00	0.00	0.00
Top flg spl	-20.78	0.00	0.00	0.00
Bot flg spl	-20.78	0.00	0.00	0.00
8	-21.25	0.00	0.00	0.00
9	-21.61	0.00	0.00	0.00
10	-32.89	0.00	0.00	0.00

WAITSFIELD
Girder 5 : Rating Output : Service Shear
Mon Dec 21 09:21:15 2015

Girder 5 Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	103.09	7.58	19.38	70.32	0.37	43.28	27.72
	8.60	89.10	6.80	17.37	66.07	0.61	39.96	25.45
1	17.20	76.73	6.03	15.42	62.26	0.00	37.11	24.12
	25.80	66.00	5.29	13.56	59.37	-1.45	35.15	23.73
2	34.40	56.89	4.57	11.67	55.96	-3.74	32.89	24.27
	43.00	46.75	3.82	9.76	52.04	-7.47	30.34	24.74
Top flg spl		44.40	3.64	9.31	50.86	-9.40	29.60	25.08
Bot flg spl		44.40	3.64	9.31	50.86	-9.40	29.60	25.08
3	51.60	36.45	3.06	7.83	47.59	-12.44	27.50	25.46
	60.20	25.89	2.30	5.88	43.23	-15.85	24.71	25.80
4	68.80	15.11	1.54	3.93	38.75	-18.63	21.85	25.98
	77.40	8.59	0.78	1.03	33.19	-21.58	16.09	24.70
5	86.00	0.37	0.02	0.05	28.45	-24.85	13.51	23.90
	94.60	-9.55	-0.74	-1.93	25.13	-28.42	15.44	23.56
6	103.20	-21.17	-1.51	-3.85	21.45	-32.29	-17.90	23.70
	111.80	-31.99	-2.27	-5.81	17.26	-36.88	-20.99	23.23
7	120.40	-42.65	-3.04	-7.76	12.95	-42.22	-24.44	23.21
Top flg spl		-50.73	-3.62	-9.26	9.56	-46.83	-27.32	23.49
Bot flg spl		-50.73	-3.62	-9.26	9.56	-46.83	-27.32	23.49
	129.00	-53.18	-3.79	-9.69	7.82	-48.11	-27.92	23.48
8	137.60	-63.50	-4.56	-11.65	3.95	-55.15	-32.41	24.49
	146.20	-73.62	-5.34	-13.65	1.48	-63.36	-38.98	26.22
9	154.80	-83.53	-6.14	-15.69	0.00	-72.73	-44.40	28.69
	163.40	-93.49	-6.95	-17.75	-0.49	-79.59	-48.68	31.44
10	172.00	-103.41	-7.77	-19.85	0.00	-85.16	-51.81	34.62

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	27.99	0.00	0.00	0.00
1	24.12	0.00	0.00	0.00
2	22.22	0.00	0.00	0.00
Top flg spl	20.93	0.00	0.00	0.00
Bot flg spl	20.93	0.00	0.00	0.00
3	20.03	0.00	0.00	0.00
4	17.33	0.00	0.00	0.00
5	12.23	0.00	0.00	0.00
6	-14.93	0.00	0.00	0.00
7	-18.61	0.00	0.00	0.00
Top flg spl	-20.21	0.00	0.00	0.00
Bot flg spl	-20.21	0.00	0.00	0.00
8	-23.01	0.00	0.00	0.00
9	-28.69	0.00	0.00	0.00
10	-34.62	0.00	0.00	0.00

WAITSFIELD
Girder 1 : Rating Output : Lrfd Ratings
Mon Dec 21 09:18:31 2015

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Bending Oper.	Shear Inv.	Shear Oper.
0.00	11003. C	928.39	112.31 B	196.54	5.08	8.89
17.20	13447. C	928.39	4.69 B	8.21	6.45	11.29
34.40	13447. C	928.39	2.17 B	3.80	8.61	15.06
45.00L	13447. C	928.39	1.61 B	2.83	10.28	17.99
45.00R	15422. C	928.39	2.02 B	3.54	10.28	17.99
51.60	15422. C	928.39	1.78 B	3.11	11.64	20.36
68.80	15422. C	928.39	1.44 B	2.52	15.65	27.39
86.00	15422. C	928.39	1.36 B	2.38	17.04	29.82
103.20	15422. C	928.39	1.50 B	2.63	13.04	22.82
120.40	15422. C	928.39	1.94 B	3.40	10.21	17.86
127.00L	15422. C	928.39	2.24 B	3.93	9.43	16.51
127.00R	13447. C	928.39	1.79 B	3.14	9.43	16.51
137.60	13447. C	928.39	2.48 B	4.33	8.37	14.65
154.80	13447. C	928.39	5.62 B	9.84	7.23	12.66
172.00	13447. C	928.39	566.97 B	992.20	6.18	10.81

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Bending Oper.
0.00	47500. S	151.93 B	197.51
17.20	47500. S	4.97 B	6.47
34.40	47500. S	2.18 B	2.83
51.60	47500. S	2.10 B	2.73
68.80	47500. S	1.69 B	2.19
86.00	47500. S	1.59 B	2.07
103.20	47500. S	1.76 B	2.29
120.40	47500. S	2.30 B	2.98
137.60	47500. S	2.49 B	3.23

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154.80           47500. S                    5.96 B   7.75
172.00           47500. S                  627.41 B 815.63

   ****************************************************
   Minimum rating is  1.36 at location  86.00 in span 1.
   ****************************************************


        Rating Codes:
        T - Top steel governs
        B - Bottom steel governs
        C - Concrete governs
        R - Rebar governs
        V - Shear governs
        S - Serviceability governs

        Mom Strength Codes:
        C - Compact
        B - Braced non-compact
        U - Unbraced non-compact
        T - Transition between compact and braced non-compact
        S - Serviceability

   Noncompact shapes ratings based on stress, as

                Fb  -  factored dead load stress
           IR = --------------------------------
                       factored LL+I stress
```

WAITSFIELD
Girder 2 : Rating Output : Lrfd Ratings
Mon Dec 21 09:18:44 2015

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Bending Oper.	Shear Inv.	Shear Oper.
0.00	11225. C	928.39	125.18 B	219.06	4.36	7.64
17.20	14218. C	1241.60	5.08 B	8.88	8.97	15.69
34.40	14218. C	1241.60	2.46 B	4.31	10.93	19.12
45.00L	14218. C	1170.25	1.84 B	3.22	11.27	19.73
45.00R	16800. C	1170.25	2.42 B	4.23	11.27	19.73
51.60	16800. C	1241.60	2.13 B	3.73	12.78	22.36
68.80	16800. C	1241.60	1.72 B	3.00	15.32	26.81
86.00	16800. C	1241.60	1.60 B	2.79	17.66	30.90
103.20	16800. C	1241.60	1.70 B	2.98	14.60	25.55
120.40	16800. C	1241.60	2.11 B	3.70	12.23	21.41
127.00L	16800. C	1170.25	2.39 B	4.19	10.79	18.88
127.00R	14218. C	1170.25	1.83 B	3.20	10.79	18.88
137.60	14218. C	1241.60	2.44 B	4.27	10.47	18.31
154.80	14218. C	1241.60	5.11 B	8.94	8.88	15.53
172.00	14218. C	928.39	252.60 B	442.05	4.57	8.00

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Bending Oper.
0.00	47500. S	168.19 B	218.65
17.20	47500. S	5.07 B	6.60
34.40	47500. S	2.28 B	2.96
51.60	47500. S	2.21 B	2.87
68.80	47500. S	1.74 B	2.26
86.00	47500. S	1.60 B	2.08
103.20	47500. S	1.72 B	2.24
120.40	47500. S	2.19 B	2.84
137.60	47500. S	2.26 B	2.94

```
154.80           47500. S                    5.11 B   6.64
172.00           47500. S                  267.91 B 348.29

   ****************************************************
   Minimum rating is  1.60 at location  86.00 in span 1.
   ****************************************************


        Rating Codes:
        T - Top steel governs
        B - Bottom steel governs
        C - Concrete governs
        R - Rebar governs
        V - Shear governs
        S - Serviceability governs

        Mom Strength Codes:
        C - Compact
        B - Braced non-compact
        U - Unbraced non-compact
        T - Transition between compact and braced non-compact
        S - Serviceability

   Noncompact shapes ratings based on stress, as

                Fb  -  factored dead load stress
           IR = --------------------------------
                      factored LL+I stress
```

WAITSFIELD
Girder 3 : Rating Output : Lrfd Ratings
Mon Dec 21 09:18:50 2015

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Rating Factors Bending Oper.	Rating Factors Shear Inv.	Rating Factors Shear Oper.
0.00	11225. C	928.39	194.21 B	339.87	4.73	8.29
17.20	14218. C	1241.60	5.48 B	9.58	9.68	16.94
34.40	14218. C	1241.60	2.67 B	4.68	11.63	20.35
45.00L	14218. C	1170.25	2.02 B	3.53	11.96	20.93
45.00R	16800. C	1170.25	2.65 B	4.64	11.96	20.93
51.60	16800. C	1241.60	2.35 B	4.11	13.53	23.68
68.80	16800. C	1241.60	1.92 B	3.36	15.75	27.56
86.00	16800. C	1241.60	1.80 B	3.15	18.59	32.53
103.20	16800. C	1241.60	1.92 B	3.36	15.76	27.58
120.40	16800. C	1241.60	2.35 B	4.11	13.52	23.67
127.00L	16800. C	1170.25	2.65 B	4.63	11.94	20.90
127.00R	14218. C	1170.25	2.02 B	3.53	11.94	20.90
137.60	14218. C	1241.60	2.67 B	4.67	11.60	20.30
154.80	14218. C	1241.60	5.47 B	9.57	9.64	16.87
172.00	11225. C	928.39	192.23 B	336.40	4.74	8.30

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Rating Factors Bending Oper.
0.00	47500. S	260.92 B	339.19
17.20	47500. S	5.47 B	7.11
34.40	47500. S	2.47 B	3.21
51.60	47500. S	2.43 B	3.16
68.80	47500. S	1.94 B	2.52
86.00	47500. S	1.80 B	2.33
103.20	47500. S	1.94 B	2.52
120.40	47500. S	2.43 B	3.15
137.60	47500. S	2.46 B	3.20

```
154.80           47500. S                    5.46 B   7.09
172.00           47500. S                  258.25 B 335.73

   ****************************************************
   Minimum rating is  1.80 at location  86.00 in span 1.
   ****************************************************


        Rating Codes:
        T - Top steel governs
        B - Bottom steel governs
        C - Concrete governs
        R - Rebar governs
        V - Shear governs
        S - Serviceability governs

        Mom Strength Codes:
        C - Compact
        B - Braced non-compact
        U - Unbraced non-compact
        T - Transition between compact and braced non-compact
        S - Serviceability

   Noncompact shapes ratings based on stress, as

                Fb  -  factored dead load stress
           IR = --------------------------------
                      factored LL+I stress
```

WAITSFIELD
Girder 4 : Rating Output : Lrfd Ratings
Mon Dec 21 09:18:59 2015

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Rating Factors Bending Oper.	Rating Factors Shear Inv.	Rating Factors Shear Oper.
0.00	14218. C	928.39	255.18 B	446.56	4.57	8.00
17.20	14218. C	1241.60	5.12 B	8.96	8.87	15.51
34.40	14218. C	1241.60	2.44 B	4.28	10.49	18.36
45.00L	14218. C	1170.25	1.83 B	3.20	10.82	18.93
45.00R	16800. C	1170.25	2.40 B	4.20	10.82	18.93
51.60	16800. C	1241.60	2.11 B	3.70	12.25	21.44
68.80	16800. C	1241.60	1.71 B	2.99	14.62	25.58
86.00	16800. C	1241.60	1.60 B	2.79	18.76	32.83
103.20	16800. C	1241.60	1.72 B	3.00	15.29	26.75
120.40	16800. C	1241.60	2.13 B	3.72	12.77	22.34
127.00L	16800. C	1170.25	2.41 B	4.22	11.25	19.68
127.00R	14218. C	1170.25	1.84 B	3.22	11.25	19.68
137.60	14218. C	1241.60	2.46 B	4.30	10.89	19.06
154.80	14218. C	1241.60	5.07 B	8.87	9.08	15.89
172.00	11225. C	928.39	124.12 B	217.20	4.37	7.65

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Rating Factors Bending Oper.
0.00	47500. S	270.65 B	351.84
17.20	47500. S	5.12 B	6.66
34.40	47500. S	2.27 B	2.95
51.60	47500. S	2.19 B	2.85
68.80	47500. S	1.73 B	2.24
86.00	47500. S	1.60 B	2.08
103.20	47500. S	1.73 B	2.25
120.40	47500. S	2.20 B	2.86
137.60	47500. S	2.27 B	2.95

```
154.80           47500. S                    5.06 B   6.58
172.00           47500. S                  166.76 B 216.79

   ****************************************************
   Minimum rating is  1.60 at location  86.00 in span 1.
   ****************************************************


        Rating Codes:
        T - Top steel governs
        B - Bottom steel governs
        C - Concrete governs
        R - Rebar governs
        V - Shear governs
        S - Serviceability governs

        Mom Strength Codes:
        C - Compact
        B - Braced non-compact
        U - Unbraced non-compact
        T - Transition between compact and braced non-compact
        S - Serviceability

   Noncompact shapes ratings based on stress, as

                Fb  -  factored dead load stress
           IR = --------------------------------
                      factored LL+I stress
```

WAITSFIELD
Girder 5 : Rating Output : Lrfd Ratings
Mon Dec 21 09:19:08 2015

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Bending Oper.	Shear Inv.	Shear Oper.
0.00	13447. C	928.39	571.38 B	999.92	6.18	10.82
17.20	13447. C	928.39	5.64 B	9.87	7.36	12.88
34.40	13447. C	928.39	2.48 B	4.35	8.52	14.90
45.00L	13447. C	928.39	1.80 B	3.15	9.61	16.82
45.00R	15422. C	928.39	2.25 B	3.94	9.61	16.82
51.60	15422. C	928.39	1.95 B	3.41	10.41	18.22
68.80	15422. C	928.39	1.51 B	2.64	13.30	23.27
86.00	15422. C	928.39	1.36 B	2.38	18.64	32.61
103.20	15422. C	928.39	1.44 B	2.52	15.82	27.69
120.40	15422. C	928.39	1.78 B	3.11	11.63	20.36
127.00L	15422. C	928.39	2.02 B	3.54	10.25	17.94
127.00R	13447. C	928.39	1.61 B	2.83	10.25	17.94
137.60	13447. C	928.39	2.17 B	3.80	8.56	14.97
154.80	13447. C	928.39	4.70 B	8.22	6.23	10.90
172.00	11003. C	928.39	110.28 B	192.99	5.10	8.92

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Bending Oper.
0.00	47500. S	632.29 B	821.98
17.20	47500. S	5.99 B	7.78
34.40	47500. S	2.50 B	3.25
51.60	47500. S	2.30 B	3.00
68.80	47500. S	1.77 B	2.30
86.00	47500. S	1.59 B	2.07
103.20	47500. S	1.69 B	2.19
120.40	47500. S	2.10 B	2.73
137.60	47500. S	2.18 B	2.83

```
154.80           47500. S                    4.98 B   6.47
172.00           47500. S                  149.19 B 193.95

   ****************************************************
   Minimum rating is  1.36 at location  86.00 in span 1.
   ****************************************************


        Rating Codes:
        T - Top steel governs
        B - Bottom steel governs
        C - Concrete governs
        R - Rebar governs
        V - Shear governs
        S - Serviceability governs

        Mom Strength Codes:
        C - Compact
        B - Braced non-compact
        U - Unbraced non-compact
        T - Transition between compact and braced non-compact
        S - Serviceability

   Noncompact shapes ratings based on stress, as

                Fb  -  factored dead load stress
           IR = --------------------------------
                      factored LL+I stress
```

WAITSFIELD
Girder 1 : Rating Output : Shear Connectors
Mon Dec 21 09:22:12 2015

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 7.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	25.99	1803.3	132175.	2	21.80
	8.60	23.59	1803.3	132175.	2	24.03 *
1	17.20	21.51	1803.3	132175.	2	26.35 *
	25.80	19.77	1803.3	132175.	2	28.67 *
2	34.40	18.36	1803.3	132175.	2	30.87 *
	43.00	18.19	1803.3	132175.	2	31.16 *
3	51.60	18.97	2021.1	160309.	2	32.34 *
	60.20	19.21	2045.9	160309.	2	31.54 *
4	68.80	19.41	2045.9	160309.	2	31.21 *
	77.40	19.59	2045.9	160309.	2	30.93 *
5	86.00	19.68	2045.9	160309.	2	30.79 *
	94.60	19.68	2045.9	160309.	2	30.79 *
6	103.20	19.59	2045.9	160309.	2	30.93 *
	111.80	19.41	2045.9	160309.	2	31.21 *
7	120.40	19.23	2045.9	160309.	2	31.51 *
	129.00	18.96	1818.6	132175.	2	29.64 *
8	137.60	18.85	1803.3	132175.	2	30.07 *
	146.20	18.88	1803.3	132175.	2	30.02 *
9	154.80	19.05	1803.3	132175.	2	29.75 *
	163.40	19.73	1803.3	132175.	2	28.73 *
10	172.00	20.78	1803.3	132175.	2	27.28 *

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1
71 between tenth points 0 and 5
71 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

WAITSFIELD
Girder 2 : Rating Output : Shear Connectors
Mon Dec 21 09:22:03 2015

Girder 2 Shear Connectors

Maximum Pitch for 0.875 X 7.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	24.57	1967.7	138719.	2	22.18
	8.60	19.92	1967.7	138719.	2	27.37 *
1	17.20	17.50	1967.7	138719.	2	31.15 *
	25.80	17.33	1967.7	138719.	2	31.45 *
2	34.40	19.40	1967.7	138719.	2	26.98 *
	43.00	20.79	1967.7	138719.	2	25.30 *
3	51.60	22.47	2222.9	168634.	2	25.41 *
	60.20	22.88	2245.4	168634.	2	24.74 *
4	68.80	22.55	2245.4	168634.	2	25.33 *
	77.40	22.42	2245.4	168634.	2	25.48 *
5	86.00	22.32	2245.4	168634.	2	25.73 *
	94.60	22.27	2245.4	168634.	2	25.78 *
6	103.20	22.26	2245.4	168634.	2	26.08 *
	111.80	22.31	2245.4	168634.	2	26.03 *
7	120.40	22.14	2245.4	168634.	2	26.23 *
	129.00	21.53	1981.8	138719.	2	25.14 *
8	137.60	21.14	1967.7	138719.	2	25.79 *
	146.20	20.97	1967.7	138719.	2	25.99 *
9	154.80	21.03	1967.7	138719.	2	25.92 *
	163.40	23.08	1967.7	138719.	2	23.61
10	172.00	26.55	1967.7	138719.	2	20.53

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1
85 between tenth points 0 and 5
85 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

WAITSFIELD
Girder 3 : Rating Output : Shear Connectors
Mon Dec 21 09:21:50 2015

Girder 3 Shear Connectors

Maximum Pitch for 0.875 X 7.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	19.17	1967.7	138719.	2	28.43 *
	8.60	15.72	1967.7	138719.	2	34.68 *
1	17.20	13.74	1967.7	138719.	2	39.67 *
	25.80	13.24	1967.7	138719.	2	41.18 *
2	34.40	14.21	1967.7	138719.	2	29.71 *
	43.00	15.50	1967.7	138719.	2	28.15 *
3	51.60	17.49	2222.9	168634.	2	27.76 *
	60.20	18.26	2245.4	168634.	2	26.75 *
4	68.80	18.45	2245.4	168634.	2	29.06 *
	77.40	17.91	2245.4	168634.	2	29.80 *
5	86.00	17.72	2245.4	168634.	2	32.18 *
	94.60	17.88	2245.4	168634.	2	31.90 *
6	103.20	18.40	2245.4	168634.	2	31.56 *
	111.80	18.08	2245.4	168634.	2	32.11 *
7	120.40	17.55	2245.4	168634.	2	33.10 *
	129.00	16.57	1981.8	138719.	2	32.66 *
8	137.60	15.82	1967.7	138719.	2	34.46 *
	146.20	15.27	1967.7	138719.	2	35.69 *
9	154.80	14.95	1967.7	138719.	2	36.47 *
	163.40	16.42	1967.7	138719.	2	33.21 *
10	172.00	19.16	1967.7	138719.	2	28.46 *

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1
85 between tenth points 0 and 5
85 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

WAITSFIELD
Girder 4 : Rating Output : Shear Connectors
Mon Dec 21 09:21:40 2015

Girder 4 Shear Connectors

Maximum Pitch for 0.875 X 7.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	26.56	1967.7	138719.	2	20.52
	8.60	22.14	1967.7	138719.	2	24.63 *
1	17.20	19.32	1967.7	138719.	2	28.22 *
	25.80	18.10	1967.7	138719.	2	30.12 *
2	34.40	18.49	1967.7	138719.	2	28.33 *
	43.00	19.65	1967.7	138719.	2	26.77 *
3	51.60	21.86	2222.9	168634.	2	26.37 *
	60.20	22.40	2245.4	168634.	2	25.50 *
4	68.80	22.19	2245.4	168634.	2	25.44 *
	77.40	21.16	2245.4	168634.	2	26.61 *
5	86.00	20.85	2245.4	168634.	2	27.32 *
	94.60	21.27	2245.4	168634.	2	26.81 *
6	103.20	22.41	2245.4	168634.	2	25.91 *
	111.80	22.59	2245.4	168634.	2	25.71 *
7	120.40	22.54	2245.4	168634.	2	25.76 *
	129.00	22.25	1981.8	138719.	2	24.32 *
8	137.60	21.78	1967.7	138719.	2	25.03 *
	146.20	21.12	1967.7	138719.	2	25.81 *
9	154.80	20.28	1967.7	138719.	2	26.87 *
	163.40	21.70	1967.7	138719.	2	25.12 *
10	172.00	24.57	1967.7	138719.	2	22.19

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1
85 between tenth points 0 and 5
85 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

WAITSFIELD
Girder 5 : Rating Output : Shear Connectors
Mon Dec 21 09:21:31 2015

Girder 5 Shear Connectors

Maximum Pitch for 0.875 X 7.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	20.79	1803.3	132175.	2	27.26 *
	8.60	19.09	1803.3	132175.	2	29.69 *
1	17.20	18.09	1803.3	132175.	2	31.33 *
	25.80	17.80	1803.3	132175.	2	31.85 *
2	34.40	18.21	1803.3	132175.	2	31.13 *
	43.00	18.55	1803.3	132175.	2	30.55 *
3	51.60	19.09	2021.1	160309.	2	32.12 *
	60.20	19.35	2045.9	160309.	2	31.31 *
4	68.80	19.49	2045.9	160309.	2	31.09 *
	77.40	18.53	2045.9	160309.	2	32.70 *
5	86.00	17.92	2045.9	160309.	2	33.81 *
	94.60	17.67	2045.9	160309.	2	34.29 *
6	103.20	17.78	2045.9	160309.	2	34.08 *
	111.80	17.42	2045.9	160309.	2	34.77 *
7	120.40	17.40	2045.9	160309.	2	34.81 *
	129.00	17.61	1818.6	132175.	2	31.91 *
8	137.60	18.36	1803.3	132175.	2	30.86 *
	146.20	19.67	1803.3	132175.	2	28.82 *
9	154.80	21.51	1803.3	132175.	2	26.34 *
	163.40	23.58	1803.3	132175.	2	24.04 *
10	172.00	25.97	1803.3	132175.	2	21.83

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1
71 between tenth points 0 and 5
71 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

WAITSFIELD
Girder 1 : Rating Output : Deflections
Mon Dec 21 09:16:06 2015

Girder 1 Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Down	Live+I Max Up
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1.787	0.584	1.203	0.088	0.226	0.358	0.000
1	3.466	1.132	2.334	0.172	0.438	0.695	0.000
	5.080	1.660	3.420	0.251	0.643	1.022	0.000
2	6.503	2.126	4.377	0.322	0.823	1.311	0.000
	7.736	2.530	5.206	0.383	0.979	1.561	0.000
3	8.778	2.871	5.907	0.435	1.112	1.774	0.000
	9.593	3.139	6.454	0.475	1.215	1.940	0.000
4	10.192	3.335	6.857	0.505	1.291	2.060	0.000
	10.551	3.453	7.097	0.523	1.337	2.126	0.000
5	10.664	3.491	7.173	0.529	1.351	2.141	0.000
	10.527	3.446	7.081	0.522	1.334	2.105	0.000
6	10.154	3.324	6.831	0.503	1.286	2.020	0.000
	9.532	3.120	6.413	0.472	1.207	1.883	0.000
7	8.703	2.848	5.856	0.431	1.102	1.708	0.000
	7.653	2.503	5.150	0.379	0.968	1.489	0.000
8	6.423	2.101	4.323	0.318	0.812	1.241	0.000
	4.978	1.628	3.351	0.246	0.629	0.958	0.000
9	3.426	1.120	2.306	0.169	0.433	0.657	0.000
	1.767	0.577	1.189	0.087	0.223	0.337	0.000
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Influence surfaces for defl developed assuming unequal
participation of girders in resisting deflection.

	Loc							
Brace	10.25	2.117	0.692	1.426	0.105	0.268	0.424	0.000
Brace	28.25	5.505	1.799	3.706	0.272	0.696	1.108	0.000
Brace	46.25	8.163	2.670	5.493	0.404	1.034	1.649	0.000
Brace	64.25	9.910	3.243	6.667	0.491	1.255	2.004	0.000
Brace	82.25	10.645	3.484	7.160	0.528	1.349	2.140	0.000
Brace	100.25	10.309	3.374	6.934	0.511	1.306	2.055	0.000
Brace	118.25	8.930	2.922	6.008	0.442	1.131	1.755	0.000
Web Spl	122.00	8.526	2.790	5.736	0.422	1.079	1.671	0.000
Brace	136.25	6.628	2.168	4.460	0.328	0.838	1.282	0.000
Brace	154.25	3.528	1.153	2.375	0.174	0.446	0.677	0.000

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

WAITSFIELD
Girder 2 : Rating Output : Deflections
Mon Dec 21 09:16:21 2015

Girder 2 Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Down	Live+I Max Up
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1.878	0.603	1.275	0.085	0.218	0.299	0.000
1	3.642	1.170	2.471	0.165	0.422	0.580	0.000
	5.329	1.713	3.616	0.242	0.619	0.850	0.000
2	6.823	2.195	4.628	0.310	0.793	1.093	0.000
	8.124	2.615	5.509	0.370	0.945	1.308	0.000
3	9.231	2.973	6.258	0.421	1.075	1.496	0.000
	10.099	3.254	6.845	0.460	1.177	1.648	0.000
4	10.744	3.464	7.280	0.490	1.253	1.766	0.000
	11.131	3.590	7.541	0.508	1.298	1.834	0.000
5	11.259	3.632	7.627	0.514	1.314	1.858	0.000
	11.122	3.588	7.533	0.508	1.298	1.837	0.000
6	10.735	3.464	7.272	0.490	1.253	1.772	0.000
	10.086	3.254	6.832	0.461	1.177	1.659	0.000
7	9.216	2.972	6.243	0.421	1.075	1.509	0.000
	8.108	2.614	5.494	0.370	0.946	1.318	0.000
8	6.808	2.195	4.614	0.311	0.794	1.100	0.000
	5.278	1.701	3.577	0.241	0.615	0.851	0.000
9	3.633	1.170	2.463	0.165	0.423	0.585	0.000
	1.873	0.603	1.270	0.085	0.218	0.301	0.000
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Influence surfaces for defl developed assuming unequal
participation of girders in resisting deflection.

	Loc							
Brace	10.25	2.226	0.715	1.511	0.101	0.258	0.354	0.000
Brace	17.75	3.750	1.205	2.545	0.170	0.435	0.598	0.000
Brace	28.25	5.774	1.857	3.918	0.262	0.670	0.922	0.000
Brace	35.75	7.040	2.265	4.775	0.320	0.818	1.129	0.000
Brace	46.25	8.578	2.762	5.816	0.391	0.998	1.385	0.000
Brace	53.75	9.469	3.050	6.419	0.431	1.103	1.537	0.000
Brace	64.25	10.441	3.365	7.075	0.476	1.217	1.712	0.000
Brace	71.75	10.903	3.516	7.388	0.498	1.271	1.794	0.000
Brace	82.25	11.235	3.624	7.611	0.513	1.311	1.853	0.000
Brace	89.75	11.233	3.624	7.609	0.513	1.311	1.854	0.000
Brace	100.25	10.896	3.515	7.380	0.498	1.272	1.799	0.000
Brace	107.75	10.430	3.365	7.065	0.476	1.217	1.719	0.000
Brace	118.25	9.454	3.050	6.405	0.432	1.103	1.550	0.000
Web Spl	122.00	9.029	2.912	6.117	0.412	1.054	1.477	0.000
Brace	125.75	8.562	2.761	5.801	0.391	0.999	1.398	0.000
Brace	136.25	7.025	2.265	4.761	0.320	0.819	1.136	0.000
Brace	143.75	5.761	1.857	3.905	0.263	0.671	0.928	0.000
Brace	154.25	3.741	1.205	2.536	0.170	0.436	0.602	0.000
Brace	161.75	2.220	0.715	1.505	0.101	0.258	0.357	0.000

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

WAITSFIELD
Girder 3 : Rating Output : Deflections
Mon Dec 21 09:16:31 2015

Girder 3 Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Down	Live+I Max Up
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1.903	0.603	1.300	0.084	0.216	0.261	0.000
1	3.689	1.170	2.520	0.164	0.419	0.508	0.000
	5.400	1.713	3.687	0.240	0.613	0.745	0.000
2	6.915	2.194	4.721	0.307	0.786	0.957	0.000
	8.235	2.614	5.620	0.366	0.936	1.145	0.000
3	9.359	2.972	6.386	0.417	1.065	1.308	0.000
	10.242	3.254	6.988	0.456	1.165	1.438	0.000
4	10.899	3.464	7.435	0.485	1.240	1.538	0.000
	11.294	3.590	7.705	0.503	1.286	1.596	0.000
5	11.426	3.632	7.794	0.509	1.301	1.615	0.000
	11.289	3.588	7.701	0.503	1.285	1.596	0.000
6	10.899	3.464	7.435	0.485	1.241	1.538	0.000
	10.242	3.254	6.988	0.456	1.165	1.438	0.000
7	9.360	2.973	6.387	0.417	1.065	1.308	0.000
	8.236	2.615	5.621	0.366	0.936	1.145	0.000
8	6.917	2.195	4.722	0.307	0.786	0.958	0.000
	5.362	1.701	3.661	0.238	0.609	0.740	0.000
9	3.691	1.170	2.521	0.164	0.419	0.508	0.000
	1.904	0.603	1.300	0.084	0.216	0.261	0.000
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Influence surfaces for defl developed assuming unequal
participation of girders in resisting deflection.

	Loc							
Brace	10.25	2.254	0.715	1.540	0.100	0.256	0.310	0.000
Brace	17.75	3.800	1.205	2.595	0.169	0.431	0.523	0.000
Brace	28.25	5.851	1.856	3.995	0.260	0.664	0.808	0.000
Brace	35.75	7.135	2.264	4.871	0.317	0.811	0.988	0.000
Brace	46.25	8.696	2.761	5.934	0.387	0.989	1.212	0.000
Brace	53.75	9.601	3.050	6.551	0.427	1.092	1.343	0.000
Brace	64.25	10.590	3.365	7.225	0.472	1.205	1.492	0.000
Brace	71.75	11.061	3.515	7.546	0.493	1.259	1.562	0.000
Brace	82.25	11.401	3.624	7.777	0.508	1.298	1.612	0.000
Brace	89.75	11.401	3.624	7.777	0.508	1.298	1.612	0.000
Brace	100.25	11.062	3.516	7.546	0.493	1.259	1.562	0.000
Brace	107.75	10.590	3.365	7.225	0.472	1.205	1.492	0.000
Brace	118.25	9.602	3.050	6.552	0.427	1.092	1.343	0.000
Web Spl	122.00	9.171	2.912	6.258	0.408	1.043	1.281	0.000
Brace	125.75	8.697	2.762	5.935	0.387	0.989	1.212	0.000
Brace	136.25	7.137	2.265	4.872	0.317	0.811	0.989	0.000
Brace	143.75	5.853	1.857	3.996	0.260	0.664	0.808	0.000
Brace	154.25	3.801	1.205	2.596	0.169	0.431	0.523	0.000
Brace	161.75	2.256	0.715	1.541	0.100	0.256	0.310	0.000

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

WAITSFIELD
Girder 4 : Rating Output : Deflections
Mon Dec 21 09:16:40 2015

Girder 4 Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Down	Live+I Max Up
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1.872	0.603	1.270	0.085	0.218	0.301	0.000
1	3.630	1.169	2.462	0.165	0.423	0.585	0.000
	5.314	1.711	3.602	0.242	0.619	0.855	0.000
2	6.805	2.193	4.613	0.310	0.793	1.100	0.000
	8.105	2.612	5.493	0.370	0.945	1.318	0.000
3	9.213	2.970	6.243	0.421	1.075	1.509	0.000
	10.083	3.251	6.832	0.461	1.177	1.659	0.000
4	10.732	3.461	7.271	0.490	1.253	1.772	0.000
	11.124	3.587	7.537	0.508	1.298	1.837	0.000
5	11.256	3.629	7.627	0.514	1.314	1.858	0.000
	11.124	3.585	7.538	0.508	1.298	1.834	0.000
6	10.742	3.461	7.281	0.490	1.253	1.766	0.000
	10.097	3.252	6.845	0.460	1.177	1.648	0.000
7	9.229	2.970	6.258	0.421	1.075	1.496	0.000
	8.122	2.613	5.510	0.370	0.945	1.308	0.000
8	6.823	2.193	4.629	0.310	0.793	1.093	0.000
	5.290	1.699	3.590	0.240	0.614	0.845	0.000
9	3.642	1.169	2.472	0.165	0.422	0.580	0.000
	1.879	0.603	1.276	0.085	0.218	0.299	0.000
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Influence surfaces for defl developed assuming unequal
participation of girders in resisting deflection.

	Loc							
Brace	10.25	2.218	0.714	1.504	0.101	0.258	0.357	0.000
Brace	17.75	3.739	1.204	2.535	0.170	0.436	0.602	0.000
Brace	28.25	5.758	1.855	3.903	0.263	0.671	0.928	0.000
Brace	35.75	7.022	2.263	4.759	0.320	0.819	1.136	0.000
Brace	46.25	8.559	2.759	5.800	0.391	0.999	1.397	0.000
Brace	53.75	9.451	3.047	6.404	0.432	1.103	1.550	0.000
Brace	64.25	10.427	3.362	7.065	0.476	1.217	1.719	0.000
Brace	71.75	10.893	3.513	7.380	0.498	1.272	1.799	0.000
Brace	82.25	11.231	3.621	7.609	0.513	1.311	1.854	0.000
Brace	89.75	11.233	3.621	7.611	0.513	1.311	1.853	0.000
Brace	100.25	10.901	3.513	7.388	0.498	1.271	1.795	0.000
Brace	107.75	10.439	3.363	7.076	0.476	1.217	1.712	0.000
Brace	118.25	9.467	3.047	6.419	0.431	1.103	1.537	0.000
Web Spl	122.00	9.042	2.910	6.132	0.412	1.053	1.464	0.000
Brace	125.75	8.576	2.759	5.816	0.391	0.998	1.385	0.000
Brace	136.25	7.039	2.263	4.776	0.320	0.818	1.129	0.000
Brace	143.75	5.774	1.855	3.919	0.262	0.670	0.923	0.000
Brace	154.25	3.751	1.204	2.546	0.170	0.435	0.598	0.000
Brace	161.75	2.226	0.714	1.511	0.101	0.258	0.354	0.000

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

WAITSFIELD
Girder 5 : Rating Output : Deflections
Mon Dec 21 09:16:50 2015

Girder 5 Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Down	Live+I Max Up
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	1.766	0.577	1.189	0.087	0.223	0.337	0.000
1	3.424	1.119	2.305	0.169	0.432	0.657	0.000
	5.012	1.639	3.373	0.248	0.633	0.963	0.000
2	6.421	2.100	4.321	0.318	0.812	1.241	0.000
	7.651	2.503	5.148	0.379	0.968	1.488	0.000
3	8.702	2.847	5.854	0.431	1.101	1.707	0.000
	9.531	3.119	6.412	0.472	1.207	1.883	0.000
4	10.152	3.323	6.829	0.503	1.286	2.020	0.000
	10.531	3.447	7.084	0.522	1.334	2.105	0.000
5	10.664	3.491	7.173	0.529	1.351	2.141	0.000
	10.546	3.452	7.094	0.523	1.336	2.126	0.000
6	10.192	3.335	6.857	0.505	1.291	2.060	0.000
	9.593	3.139	6.454	0.475	1.215	1.939	0.000
7	8.778	2.871	5.907	0.435	1.111	1.774	0.000
	7.736	2.530	5.206	0.383	0.979	1.561	0.000
8	6.503	2.126	4.377	0.322	0.822	1.310	0.000
	5.038	1.647	3.392	0.249	0.637	1.013	0.000
9	3.466	1.132	2.334	0.171	0.438	0.695	0.000
	1.787	0.584	1.203	0.088	0.226	0.358	0.000
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Influence surfaces for defl developed assuming unequal
participation of girders in resisting deflection.

	Loc							
Brace	17.75	3.527	1.153	2.374	0.174	0.445	0.676	0.000
Brace	35.75	6.626	2.167	4.459	0.328	0.838	1.281	0.000
Brace	53.75	8.928	2.922	6.007	0.442	1.130	1.754	0.000
Brace	71.75	10.307	3.374	6.933	0.511	1.305	2.054	0.000
Brace	89.75	10.645	3.484	7.161	0.528	1.348	2.140	0.000
Brace	107.75	9.910	3.243	6.667	0.491	1.255	2.004	0.000
Web Spl	122.00	8.603	2.814	5.789	0.426	1.089	1.739	0.000
Brace	125.75	8.163	2.670	5.493	0.404	1.033	1.649	0.000
Brace	143.75	5.505	1.799	3.706	0.272	0.696	1.108	0.000
Brace	161.75	2.117	0.692	1.426	0.105	0.268	0.424	0.000

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

WAITSFIELD
Girder System : Input File : Layout/Slab/Loading Definition
Mon Dec 21 09:06:04 2015

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ID: WAITSFIELD
CONDITIONS
  ENGLISH INPUT
  ENGLISH OUTPUT
  FLOAT LANES
  GRID MODEL
  HIGH RESOLUTION MESH
  INTERMEDIATE BRACING NOT BOLTED FOR STEEL ANALYSIS
  LRFD METHOD
  RATE MODE
  RATING PROJECT
  SELF WEIGHT FOR DEAD LOAD 1
DATA
  BR-1 10.25 18. 18. 18. 18. 18. 18. 18. 18.
  BR-2 10.25 18. 18. 18. 18. 18. 18. 18. 18.
  BR-3 10.25 18. 18. 18. 18. 18. 18. 18. 18.
  BR-4 10.25 18. 18. 18. 18. 18. 18. 18. 18.
  CURB  -0.5
  FPC 4.
  GDSPC 7.5 7.5 7.5 7.5
  LANES 11. 11.
  ROADWP 31.
  SKEW-1 135. 135.
  SLABEXT 2.5 2.5
  SLABT 9.
  SLABWEAR  0.5
  SPN-1 172.
  WAC-1  0.064
  WAC-2  0.0417
  WAC-3  0.0417
  WAC-4  0.0417
  WAC-5  0.064
  WAS-1  0.0102
  WAS-2  0.0203
  WAS-3  0.0203
  WAS-4  0.0203
  WAS-5  0.0102
  WCONC 150.
  WR-1  0.23
  WR-2  0.23
  WR-3  0.23
  WR-4  0.23
  WR-5  0.23
  WS-1  0.09
  WS-2  0.09
  WS-3  0.09
  WS-4  0.09
  WS-5  0.09
GO
```

WAITSFIELD
Bracing : Rating Output : Service Forces - Girder 1 To Adjacent Girder - Brace
Mon Dec 21 09:12:45 2015

Girder 1 Forces - H L 93

Forces on Brace 11

Shear - k

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.25	0.00	-0.01	4.02	-4.97	0.00

Concurrent mom -21.19 17.94 k-ft

Moment at Girder 1 - k-ft - right side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.72	0.05	0.14	17.94	-19.18	0.00

Concurrent shear -3.67 4.02 k

Moment from fatigue trk

	7.11	7.11
Concurrent Shr	-1.45	0.00

Moment at Adjacent Girder - k-ft - left side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.70	0.03	0.07	0.00	-28.54	0.00

Concurrent shear 0.00 4.97 k

Moment from fatigue trk

	0.00	0.00
Concurrent Shr	0.00	1.45

Service Axial Forces - k

	Noncmp Dead	Comp Dead	Wearing Surface	LL+I Forces at gd 1 (+)	LL+I Forces at gd 1 (-)	LL+I Forces at adj gd (+)	LL+I Forces at adj gd (-)	Side-walk (+)	Side-walk (-)
Top chd L	0.37	0.00	-0.02	6.42	-4.02	7.13	-4.49	0.00	0.00
Top chd R	-0.18	-0.01	-0.03	5.30	-4.49	6.42	-3.77	0.00	0.00
LR-UC diag	-0.37	-0.01	-0.01	5.97	-7.38	7.38	-7.38	0.00	0.00
UC-LL diag	0.37	0.01	0.01	7.38	-5.97	7.38	-7.38	0.00	0.00
Bot chord	-0.10	0.01	0.03	0.46	-0.97	0.00	-1.68	0.00	0.00

WAITSFIELD
Bracing : Rating Output : Service Forces - Girder 2 To Adjacent Girder - Brace
Mon Dec 21 09:13:49 2015

Girder 2 Forces - H L 93

Forces on Brace 11

Shear - k

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.19	-0.01	-0.01	5.53	-2.82	0.00

Concurrent mom -19.62 15.23 k-ft

Moment at Girder 2 - k-ft - right side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.58	-0.02	-0.06	15.24	-26.80	0.00

Concurrent shear -2.77 5.49 k

Moment from fatigue trk

	6.24	7.11
Concurrent Shr	-1.13	2.16

Moment at Adjacent Girder - k-ft - left side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.47	-0.06	-0.15	0.82	-12.15	0.00

Concurrent shear -0.07 2.82 k

Moment from fatigue trk

	2.05	-1.68
Concurrent Shr	-0.01	0.93

Service Axial Forces - k

	Noncmp Dead	Comp Dead	Wearing Surface	LL+I Forces at gd 2 (+)	LL+I Forces at gd 2 (-)	LL+I Forces at adj gd (+)	LL+I Forces at adj gd (-)	Side-walk (+)	Side-walk (-)
Top chd L	0.26	0.02	0.03	2.39	-7.22	4.90	-3.81	0.00	0.00
Top chd R	-0.15	0.01	0.02	6.70	-3.81	2.39	-7.22	0.00	0.00
LR-UC diag	-0.28	-0.01	-0.01	8.20	-4.19	4.19	-8.20	0.00	0.00
UC-LL diag	0.28	0.01	0.01	4.19	-8.20	8.20	-4.19	0.00	0.00
Bot chord	-0.06	-0.01	-0.02	1.16	-0.68	1.16	0.00	0.00	0.00

WAITSFIELD
Bracing : Rating Output : Service Forces - Girder 3 To Adjacent Girder - Brace
Mon Dec 21 09:14:04 2015

Girder 3 Forces - H L 93

Forces on Brace 11

Shear - k

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.10	-0.03	-0.04	3.19	-6.15	0.00

Concurrent mom -23.66 17.42 k-ft

Moment at Girder 3 - k-ft - right side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.31	0.10	0.27	17.58	-16.46	0.00

Concurrent shear -2.75 3.19 k

Moment from fatigue trk

	6.93	7.11
Concurrent Shr	-1.12	0.00

Moment at Adjacent Girder - k-ft - left side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
-0.26	-0.07	-0.17	0.00	-31.70	0.00

Concurrent shear 0.00 6.15 k

Moment from fatigue trk

	2.05	-1.68
Concurrent Shr	0.27	1.41

Service Axial Forces - k

	Noncmp Dead	Comp Dead	Wearing Surface	LL+I Forces at gd 3 (+)	LL+I Forces at gd 3 (-)	LL+I Forces at adj gd (+)	LL+I Forces at adj gd (-)	Side-walk (+)	Side-walk (-)
Top chd L	0.14	0.04	0.02	9.14	-2.88	7.92	-4.36	0.00	0.00
Top chd R	-0.08	-0.03	-0.07	5.91	-4.40	9.14	-5.57	0.00	0.00
LR-UC diag	-0.15	-0.04	-0.06	4.73	-9.13	9.13	-9.13	0.00	0.00
UC-LL diag	0.15	0.04	0.06	9.13	-4.73	9.13	-9.13	0.00	0.00
Bot chord	-0.03	-0.01	0.02	1.38	-2.42	0.00	-2.42	0.00	0.00

WAITSFIELD
Bracing : Rating Output : Service Forces - Girder 4 To Adjacent Girder - Brace
Mon Dec 21 09:14:20 2015

Girder 4 Forces - H L 93

Forces on Brace 11

Shear - k

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.03	-0.22	-0.30	4.76	-12.53	0.00

Concurrent mom -41.96 52.56 k-ft

Moment at Girder 4 - k-ft - right side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.01	0.43	1.10	52.69	-19.49	0.00

Concurrent shear -12.50 4.70 k

Moment from fatigue trk

	21.16	21.16
Concurrent Shr	-5.09	2.12

Moment at Adjacent Girder - k-ft - left side of brace

Noncomp Dead	Comp Dead	Wearing Surface	Max LL+I	Min LL+I	Side-walk
0.18	-0.83	-2.13	0.69	-57.29	0.00

Concurrent shear -0.07 9.06 k

Moment from fatigue trk

	2.05	-1.68
Concurrent Shr	-0.01	3.71

Service Axial Forces - k

	Noncmp Dead	Comp Dead	Wearing Surface	LL+I Forces at gd 4 (+)	LL+I Forces at gd 4 (-)	LL+I Forces at adj gd (+)	LL+I Forces at adj gd (-)	Side-walk (+)	Side-walk (-)
Top chd L	-0.07	0.38	0.39	14.34	-5.43	14.32	-13.14	0.00	0.00
Top chd R	0.00	-0.11	-0.27	10.49	-13.17	14.34	-5.55	0.00	0.00
LR-UC diag	0.04	-0.33	-0.45	7.07	-18.60	18.60	-13.45	0.00	0.00
UC-LL diag	-0.04	0.33	0.45	18.60	-7.07	13.45	-18.60	0.00	0.00
Bot chord	0.04	-0.14	-0.06	0.28	-5.27	0.09	-5.27	0.00	0.00

WAITSFIELD
Bracing : Rating Output : Fatigue Stress Range Summary - Girder 1
Mon Dec 21 09:10:14 2015

Girder 1 Loading Stress Range - ksi

Brace	Type	Top Chord	Diagonal	Bottom Chord
1	C	2.13	2.52	3.71
2	C	0.64	0.89	1.01
3	C	0.50	1.07	0.81
4	C	0.46	0.99	0.60
5	C	0.50	0.94	0.45
6	C	0.50	0.87	0.39
7	C	0.46	0.73	0.27
8	C	0.41	0.58	0.12
9	C	0.37	0.71	0.49
10	C	0.45	0.64	0.76
11	B	0.52	0.86	0.36

Stresses are factored.

WAITSFIELD
Bracing : Rating Output : Fatigue Stress Range Summary - Girder 2
Mon Dec 21 09:10:45 2015

Girder 2 Loading Stress Range - ksi

Brace	Type	Top Chord	Diagonal	Bottom Chord
1	C	1.53	0.98	2.17
2	C	1.44	1.32	1.87
3	C	1.82	1.50	2.24
4	C	1.64	1.50	2.00
5	C	1.67	0.92	1.43
6	C	1.47	0.98	1.22
7	C	1.40	1.07	1.40
8	C	1.25	1.62	1.51
9	C	0.96	0.89	0.96
10	C	0.58	0.65	0.59
11	B	0.83	0.97	0.83

Stresses are factored.

WAITSFIELD
Bracing : Rating Output : Fatigue Stress Range Summary - Girder 3
Mon Dec 21 09:10:58 2015

Girder 3 Loading Stress Range - ksi

Brace	Type	Top Chord	Diagonal	Bottom Chord
1	C	1.17	0.21	1.28
2	C	1.18	0.74	1.34
3	C	1.56	0.91	1.71
4	C	1.71	0.48	1.73
5	C	1.84	0.38	1.77
6	C	1.89	0.42	1.89
7	C	1.94	0.46	1.94
8	C	1.88	0.47	1.88
9	C	1.66	0.72	1.68
10	C	1.19	1.03	1.16
11	B	0.38	0.75	0.36

Stresses are factored.

WAITSFIELD
Bracing : Rating Output : Fatigue Stress Range Summary - Girder 4
Mon Dec 21 09:12:00 2015

Girder 4 Loading Stress Range - ksi

Brace	Type	Top Chord	Diagonal	Bottom Chord
1	C	1.05	0.77	1.31
2	C	1.27	0.23	1.37
3	C	1.64	0.84	1.87
4	C	1.81	0.71	1.81
5	C	1.98	0.69	2.06
6	C	2.11	0.94	2.32
7	C	1.99	0.83	2.07
8	C	1.88	0.73	2.02
9	C	1.67	1.47	1.92
10	C	1.39	1.44	1.41
11	B	1.77	2.61	1.58

Stresses are factored.

WAITSFIELD
Bracing : Rating Output : Factored Combined Forces Summary - Girder 1
Mon Dec 21 09:08:54 2015

Girder 1 Max Combined Member Forces - k

HL-93

Brace	Type	Top Chord Tens	Top Chord Comp	Diagonal Tens	Diagonal Comp	Bottom Chord Tens	Bottom Chord Comp
1	C	9.27	-5.92	32.90	-32.90	23.36	-33.52
2	C	14.99	-12.18	18.47	-18.47	23.02	-21.60
3	C	10.77	-9.60	13.87	-13.87	17.75	-15.45
4	C	7.90	-11.06	14.75	-14.75	19.72	-10.66
5	C	5.53	-11.78	16.32	-16.32	21.36	-6.70
6	C	3.40	-11.64	15.86	-15.86	20.95	-3.58
7	C	1.63	-10.59	14.45	-14.45	19.08	-2.38
8	C	0.33	-9.73	13.93	-13.93	17.83	-2.47
9	C	2.46	-11.27	14.06	-14.06	19.44	-7.04
10	C	5.46	-11.89	9.36	-9.36	17.01	-10.68
11	B	12.92	-8.14	13.41	-13.41	0.75	-3.01

Load modifiers included.

WAITSFIELD
Bracing : Rating Output : Factored Combined Forces Summary - Girder 2
Mon Dec 21 09:09:26 2015

Girder 2 Max Combined Member Forces - k

HL-93

Brace	Type	Top Chord Tens	Top Chord Comp	Diagonal Tens	Diagonal Comp	Bottom Chord Tens	Bottom Chord Comp
1	C	1.15	-2.67	14.69	-14.69	12.68	-11.91
2	C	0.00	-23.77	24.20	-24.20	37.98	-6.03
3	C	0.00	-41.15	23.62	-23.62	54.67	-2.42
4	C	0.00	-52.03	20.70	-20.70	63.93	0.00
5	C	0.00	-56.73	20.32	-20.32	68.67	0.00
6	C	0.00	-57.04	20.94	-20.94	68.46	-3.11
7	C	0.00	-53.46	22.63	-22.63	63.92	-8.04
8	C	0.00	-46.48	23.82	-23.82	55.16	-12.67
9	C	3.61	-34.11	24.20	-24.20	38.55	-16.98
10	C	10.49	-15.72	14.17	-14.17	16.74	-14.14
11	B	11.61	-12.78	14.73	-14.73	1.94	-1.31

Load modifiers included.

WAITSFIELD
Bracing : Rating Output : Factored Combined Forces Summary - Girder 3
Mon Dec 21 09:09:39 2015

Girder 3 Max Combined Member Forces - k

HL-93

Brace	Type	Top Chord Tens	Top Chord Comp	Diagonal Tens	Diagonal Comp	Bottom Chord Tens	Bottom Chord Comp
1	C	1.64	-2.03	12.37	-12.37	11.01	-10.13
2	C	0.46	-18.60	18.12	-18.12	26.09	-1.07
3	C	0.00	-32.81	16.01	-16.01	41.74	-0.38
4	C	0.00	-44.94	21.25	-21.25	51.02	0.00
5	C	0.00	-52.90	23.31	-23.31	57.33	0.00
6	C	0.00	-57.86	24.17	-24.17	61.05	0.00
7	C	0.00	-59.50	24.52	-24.52	61.96	0.00
8	C	0.00	-56.98	24.57	-24.57	59.08	0.00
9	C	0.22	-47.09	26.63	-26.63	47.31	-0.46
10	C	1.43	-25.64	22.28	-22.28	28.19	-3.62
11	B	16.26	-9.98	16.32	-16.32	2.39	-4.25

Load modifiers included.

WAITSFIELD
Bracing : Rating Output : Factored Combined Forces Summary - Girder 4
Mon Dec 21 09:09:50 2015

Girder 4 Max Combined Member Forces - k

HL-93

Brace	Type	Top Chord Tens	Top Chord Comp	Diagonal Tens	Diagonal Comp	Bottom Chord Tens	Bottom Chord Comp
1	C	1.09	-2.01	10.59	-10.59	9.81	-5.95
2	C	8.62	-6.30	8.88	-8.88	9.11	-13.79
3	C	4.72	-7.69	9.11	-9.11	12.95	-9.35
4	C	1.42	-10.93	12.18	-12.18	17.98	-3.75
5	C	0.00	-12.85	13.73	-13.73	20.87	-0.75
6	C	0.00	-14.71	15.80	-15.80	23.99	0.00
7	C	0.00	-16.11	17.83	-17.83	26.58	0.00
8	C	0.00	-16.74	18.15	-18.15	27.36	0.00
9	C	0.14	-17.56	20.48	-20.48	29.43	-0.37
10	C	2.95	-20.08	19.30	-19.30	30.98	-3.15
11	B	26.08	-23.60	33.58	-33.58	0.36	-9.44

Load modifiers included.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

General Bridge Geometry		
Girder Spacing =	7.5	ft
Bridge Length =	172	ft
Skew =	45	degrees
Cross Slope =	2	%

Girder Info		
	CL Abut 1 Stations	Offset from Baseline (ft)
Girder 1	1341	15
Girder 2	1348.5	7.5
Girder 3	1356	0
Girder 4	1363.5	7.5
Girder 5	1371	15

Vertical Curve Data		
PVC Sta =	1310	
G1 =	0.0224	
G2 =	-0.0245	
Length =	280	ft
PVC Elevation =	728.35	ft

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

Girder 1

Station	Point Location	Deflections (in) ** Steel	Slab	SDL	Total Deflection (in)	Elevation (ft)	Vertical Curve Chord Elev (ft)	Ordinate (in)	Total Camber (in)
13+41.00	0	0	0	0	0	728.66	728.66	0.00	0.00
13+49.60	0.05	0.577	1.189	0.087	1.853	728.81	728.69	1.41	3.27
13+58.20	0.1	1.119	2.305	0.169	3.593	728.94	728.71	2.68	6.27
13+66.80	0.15	1.639	3.373	0.248	5.26	729.05	728.74	3.79	9.05
13+75.40	0.2	2.1	4.321	0.318	6.739	729.16	728.76	4.76	11.50
13+84.00	0.25	2.503	5.148	0.379	8.03	729.25	728.78	5.57	13.60
13+92.60	0.3	2.847	5.854	0.431	9.132	729.33	728.81	6.24	15.38
14+01.20	0.35	3.119	6.412	0.472	10.003	729.40	728.83	6.76	16.77
14+09.80	0.4	3.323	6.829	0.503	10.655	729.45	728.86	7.14	17.79
14+18.40	0.45	3.447	7.084	0.522	11.053	729.49	728.88	7.36	18.41
14+27.00	0.5	3.491	7.173	0.529	11.193	729.52	728.90	7.43	18.63
14+35.60	0.55	3.452	7.094	0.523	11.069	729.54	728.93	7.36	18.43
14+44.20	0.6	3.335	6.857	0.505	10.697	729.55	728.95	7.14	17.83
14+52.80	0.65	3.139	6.454	0.475	10.068	729.54	728.98	6.76	16.83
14+61.40	0.7	2.871	5.907	0.435	9.213	729.52	729.00	6.24	15.46
14+70.00	0.75	2.53	5.206	0.383	8.119	729.49	729.03	5.57	13.69
14+78.60	0.8	2.126	4.377	0.322	6.825	729.45	729.05	4.76	11.58
14+87.20	0.85	1.647	3.392	0.249	5.288	729.39	729.07	3.79	9.08
14+95.80	0.9	1.132	2.334	0.171	3.637	729.32	729.10	2.68	6.31
15+04.40	0.95	0.584	1.203	0.088	1.875	729.24	729.12	1.41	3.29
15+13.00	1	0	0	0	0	729.15	729.15	0.00	0.00

** Deflections taken from MDX design file. No future wearing surface included.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

Girder 2

Station	Point Location	Deflections (in) ** Steel	Slab	SDL	Total Deflection (in)	Elevation (ft)	Vertical Curve Chord Elev (ft)	Ordinate (in)	Total Camber (in)
13+48.50	0	0	0	0	0	728.94	728.94	0.00	0.00
13+57.10	0.05	0.603	1.27	0.085	1.958	729.07	728.95	1.41	3.37
13+65.70	0.1	1.169	2.462	0.165	3.796	729.19	728.96	2.68	6.47
13+74.30	0.15	1.711	3.602	0.242	5.555	729.29	728.98	3.79	9.35
13+82.90	0.2	2.193	4.613	0.31	7.116	729.39	728.99	4.76	11.87
13+91.50	0.25	2.612	5.493	0.37	8.475	729.47	729.00	5.57	14.05
14+00.10	0.3	2.97	6.243	0.421	9.634	729.54	729.02	6.24	15.88
14+08.70	0.35	3.251	6.832	0.461	10.544	729.60	729.03	6.76	17.31
14+17.30	0.4	3.461	7.271	0.49	11.222	729.64	729.04	7.14	18.36
14+25.90	0.45	3.587	7.537	0.508	11.632	729.67	729.06	7.36	18.99
14+34.50	0.5	3.629	7.627	0.514	11.77	729.69	729.07	7.43	19.20
14+43.10	0.55	3.585	7.538	0.508	11.631	729.70	729.08	7.36	18.99
14+51.70	0.6	3.461	7.281	0.49	11.232	729.69	729.10	7.14	18.37
14+60.30	0.65	3.252	6.845	0.46	10.557	729.67	729.11	6.76	17.32
14+68.90	0.7	2.97	6.258	0.421	9.649	729.64	729.12	6.24	15.89
14+77.50	0.75	2.613	5.51	0.37	8.493	729.60	729.14	5.57	14.07
14+86.10	0.8	2.193	4.629	0.31	7.132	729.55	729.15	4.76	11.89
14+94.70	0.85	1.699	3.59	0.24	5.529	729.48	729.16	3.79	9.32
15+03.30	0.9	1.169	2.472	0.165	3.806	729.40	729.18	2.68	6.48
15+11.90	0.95	0.603	1.276	0.085	1.964	729.31	729.19	1.41	3.38
15+20.50	1	0	0	0	0	729.20	729.20	0.00	0.00

** Deflections taken from MDX design file. No future wearing surface included.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

Girder 3

Station	Point Location	Deflections (in) ** Steel	Slab	SDL	Total Deflection (in)	Elevation (ft)	Vertical Curve Chord Elev (ft)	Ordinate (in)	Total Camber (in)
13+56.00	0	0	0	0	0	729.20	729.20	0.00	0.00
13+64.60	0.05	0.603	1.3	0.084	1.987	729.32	729.21	1.41	3.40
13+73.20	0.1	1.17	2.52	0.164	3.854	729.43	729.21	2.68	6.53
13+81.80	0.15	1.713	3.687	0.24	5.64	729.53	729.21	3.79	9.43
13+90.40	0.2	2.194	4.721	0.307	7.222	729.61	729.21	4.76	11.98
13+99.00	0.25	2.614	5.62	0.366	8.6	729.68	729.22	5.57	14.17
14+07.60	0.3	2.973	6.386	0.417	9.776	729.74	729.22	6.24	16.02
14+16.20	0.35	3.254	6.988	0.456	10.698	729.78	729.22	6.76	17.46
14+24.80	0.4	3.464	7.435	0.485	11.384	729.82	729.22	7.14	18.52
14+33.40	0.45	3.59	7.705	0.503	11.798	729.84	729.23	7.36	19.16
14+42.00	0.5	3.632	7.794	0.509	11.935	729.85	729.23	7.43	19.37
14+50.60	0.55	3.588	7.701	0.503	11.792	729.84	729.23	7.36	19.15
14+59.20	0.6	3.464	7.435	0.485	11.384	729.83	729.23	7.14	18.52
14+67.80	0.65	3.254	6.988	0.456	10.698	729.80	729.24	6.76	17.46
14+76.40	0.7	2.973	6.387	0.417	9.777	729.76	729.24	6.24	16.02
14+85.00	0.75	2.615	5.621	0.366	8.602	729.71	729.24	5.57	14.18
14+93.60	0.8	2.195	4.722	0.307	7.224	729.64	729.24	4.76	11.98
15+02.20	0.85	1.701	3.661	0.238	5.6	729.56	729.25	3.79	9.39
15+10.80	0.9	1.17	2.521	0.164	3.855	729.47	729.25	2.68	6.53
15+19.40	0.95	0.603	1.3	0.084	1.987	729.37	729.25	1.41	3.40
15+28.00	1	0	0	0	0	729.25	729.25	0.00	0.00

** Deflections taken from MDX design file. No future wearing surface included.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

Girder 4

Station	Point Location	Deflections (in) ** Steel	Slab	SDL	Total Deflection (in)	Elevation (ft)	Vertical Curve Chord Elev (ft)	Ordinate (in)	Total Camber (in)
13+63.50	0	0	0	0	0	729.16	729.16	0.00	0.00
13+72.10	0.05	0.603	1.275	0.085	1.963	729.27	729.15	1.41	3.38
13+80.70	0.1	1.17	2.471	0.165	3.806	729.37	729.14	2.68	6.48
13+89.30	0.15	1.714	3.616	0.242	5.572	729.45	729.13	3.79	9.36
13+97.90	0.2	2.195	4.628	0.31	7.133	729.52	729.13	4.76	11.89
14+06.50	0.25	2.615	5.509	0.37	8.494	729.58	729.12	5.57	14.07
14+15.10	0.3	2.973	6.258	0.421	9.652	729.63	729.11	6.24	15.90
14+23.70	0.35	3.255	6.845	0.46	10.56	729.66	729.10	6.76	17.32
14+32.30	0.4	3.464	7.28	0.49	11.234	729.69	729.09	7.14	18.37
14+40.90	0.45	3.59	7.541	0.508	11.639	729.70	729.08	7.36	19.00
14+49.50	0.5	3.632	7.627	0.514	11.773	729.70	729.08	7.43	19.21
14+58.10	0.55	3.589	7.533	0.508	11.63	729.68	729.07	7.36	18.99
14+66.70	0.6	3.464	7.272	0.49	11.226	729.65	729.06	7.14	18.36
14+75.30	0.65	3.254	6.832	0.461	10.547	729.61	729.05	6.76	17.31
14+83.90	0.7	2.973	6.243	0.421	9.637	729.56	729.04	6.24	15.88
14+92.50	0.75	2.615	5.494	0.37	8.479	729.50	729.03	5.57	14.05
15+01.10	0.8	2.195	4.614	0.311	7.12	729.42	729.03	4.76	11.88
15+09.70	0.85	1.701	3.577	0.241	5.519	729.33	729.02	3.79	9.31
15+18.30	0.9	1.17	2.463	0.165	3.798	729.23	729.01	2.68	6.47
15+26.90	0.95	0.603	1.27	0.085	1.958	729.12	729.00	1.41	3.37
15+35.50	1	0	0	0	0	728.99	728.99	0.00	0.00

** Deflections taken from MDX design file. No future wearing surface included.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

Girder 5

Station	Point Location	Deflections (in) ** Steel	Slab	SDL	Total Deflection (in)	Elevation (ft)	Vertical Curve Chord Elev (ft)	Ordinate (in)	Total Camber (in)
13+71.00	0	0	0	0	0	729.10	729.10	0.00	0.00
13+79.60	0.05	0.584	1.203	0.088	1.875	729.20	729.09	1.41	3.29
13+88.20	0.1	1.132	2.334	0.172	3.638	729.29	729.07	2.68	6.31
13+96.80	0.15	1.66	3.42	0.251	5.331	729.36	729.05	3.79	9.12
14+05.40	0.2	2.126	4.377	0.322	6.825	729.42	729.03	4.76	11.58
14+14.00	0.25	2.53	5.206	0.383	8.119	729.47	729.01	5.57	13.69
14+22.60	0.3	2.871	5.907	0.435	9.213	729.51	728.99	6.24	15.46
14+31.20	0.35	3.139	6.454	0.475	10.068	729.53	728.97	6.76	16.83
14+39.80	0.4	3.335	6.857	0.505	10.697	729.55	728.95	7.14	17.83
14+48.40	0.45	3.453	7.097	0.523	11.073	729.55	728.93	7.36	18.43
14+57.00	0.5	3.491	7.173	0.529	11.193	729.53	728.91	7.43	18.63
14+65.60	0.55	3.446	7.081	0.522	11.049	729.51	728.89	7.36	18.41
14+74.20	0.6	3.324	6.831	0.503	10.658	729.47	728.88	7.14	17.79
14+82.80	0.65	3.12	6.413	0.472	10.005	729.42	728.86	6.76	16.77
14+91.40	0.7	2.848	5.856	0.431	9.135	729.36	728.84	6.24	15.38
15+00.00	0.75	2.503	5.15	0.379	8.032	729.28	728.82	5.57	13.61
15+08.60	0.8	2.101	4.323	0.318	6.742	729.20	728.80	4.76	11.50
15+17.20	0.85	1.628	3.351	0.246	5.225	729.10	728.78	3.79	9.02
15+25.80	0.9	1.12	2.306	0.169	3.595	728.98	728.76	2.68	6.27
15+34.40	0.95	0.577	1.189	0.087	1.853	728.86	728.74	1.41	3.27
15+43.00	1	0	0	0	0	728.72	728.72	0.00	0.00

** Deflections taken from MDX design file. No future wearing surface included.

Total Deflection Comparison								
	G1	G2	G3	G4	G5	Max Difference	Max Diff G1 & G5	Max Diff G2 - G4
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.05	1.85	1.96	1.99	1.96	1.88	0.13	0.02	0.03
0.1	3.59	3.80	3.85	3.81	3.64	0.26	0.04	0.06
0.15	5.26	5.56	5.64	5.57	5.33	0.38	0.07	0.09
0.2	6.74	7.12	7.22	7.13	6.83	0.48	0.09	0.11
0.25	8.03	8.48	8.60	8.49	8.12	0.57	0.09	0.12
0.3	9.13	9.63	9.78	9.65	9.21	0.64	0.08	0.14
0.35	10.00	10.54	10.70	10.56	10.07	0.70	0.06	0.15
0.4	10.66	11.22	11.38	11.23	10.70	0.73	0.04	0.16
0.45	11.05	11.63	11.80	11.64	11.07	0.74	0.02	0.17
0.5	11.19	11.77	11.94	11.77	11.19	0.74	0.00	0.16
0.55	11.07	11.63	11.79	11.63	11.05	0.74	0.02	0.16
0.6	10.70	11.23	11.38	11.23	10.66	0.73	0.04	0.16
0.65	10.07	10.56	10.70	10.55	10.01	0.69	0.06	0.15
0.7	9.21	9.65	9.78	9.64	9.14	0.64	0.08	0.14
0.75	8.12	8.49	8.60	8.48	8.03	0.57	0.09	0.12
0.8	6.83	7.13	7.22	7.12	6.74	0.48	0.08	0.10
0.85	5.29	5.53	5.60	5.52	5.23	0.38	0.06	0.08
0.9	3.64	3.81	3.86	3.80	3.60	0.26	0.04	0.06
0.95	1.88	1.96	1.99	1.96	1.85	0.13	0.02	0.03
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Since the maximum difference among all girders is approaching 3/4 inch, consider using two separate deflection diagrams: one for the exterior girders and one for the interior girders.

Originated By: J. Olund; 12/3/2015
Checked By: T. Poulin; 12/9/2015

	Total Camber Comparison							
	G1	G2	G3	G4	G5	Max Difference	Max Diff G1 & G5	Max Diff G2 - G4
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.05	3.27	3.37	3.40	3.38	3.29	0.13	0.02	0.03
0.1	6.27	6.47	6.53	6.48	6.31	0.26	0.05	0.06
0.15	9.05	9.35	9.43	9.36	9.12	0.38	0.07	0.09
0.2	11.50	11.87	11.98	11.89	11.58	0.48	0.09	0.11
0.25	13.60	14.05	14.17	14.07	13.69	0.57	0.09	0.12
0.3	15.38	15.88	16.02	15.90	15.46	0.64	0.08	0.14
0.35	16.77	17.31	17.46	17.32	16.83	0.70	0.07	0.15
0.4	17.79	18.36	18.52	18.37	17.83	0.73	0.04	0.16
0.45	18.41	18.99	19.16	19.00	18.43	0.74	0.02	0.17
0.5	18.63	19.20	19.37	19.21	18.63	0.74	0.00	0.16
0.55	18.43	18.99	19.15	18.99	18.41	0.74	0.02	0.16
0.6	17.83	18.37	18.52	18.36	17.79	0.73	0.04	0.16
0.65	16.83	17.32	17.46	17.31	16.77	0.69	0.06	0.15
0.7	15.46	15.89	16.02	15.88	15.38	0.64	0.08	0.14
0.75	13.69	14.07	14.18	14.05	13.61	0.57	0.09	0.12
0.8	11.58	11.89	11.98	11.88	11.50	0.48	0.08	0.10
0.85	9.08	9.32	9.39	9.31	9.02	0.38	0.06	0.08
0.9	6.31	6.48	6.53	6.47	6.27	0.26	0.04	0.06
0.95	3.29	3.38	3.40	3.37	3.27	0.13	0.02	0.03
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Since the maximum difference among all girders is approaching 3/4 inch, consider using two separate camber diagrams: one for the exterior girders and one for the interior girders.

Edge Bearing Cross Frame Design Checks

Ref: AASHTO LRFD, Sixth edition with 2014 Interims

- ***Methodology***
  1. *Use maximum of: axial forces output from MDX, forces induced by wheel loads over cross frame, and wind loads.*
  2. *Analyze/Design top chord for combination of axial force plus moment from edge beam load and wheel loads as if the member acts as a simply supported beam. Assume the concrete edge beam is dead load only.*

Geometry and Properties:

- ***Steel Properties***

$F_y := 50ksi$	*Yield Strength of Members (AASHTO M270, Gr 50)*
$F_u := 65ksi$	*Ultimate Strength of Members (AASHTO Table 6.4.1-1)*
$E_s := 29000ksi$	*Steel Modulus of Elasticity*
$F_{ub} := 120ksi$	*Ultimate Tensile Strength of Bolts (AASHTO 6.4.3.1) - AASHTO M164/ASTM A325, Type 3*
$F_{exx} := 70ksi$	*Strength of Weld Metal, Assumed*
$G := 11.2ksi$	*Shear modulus of Steel*

- ***Girder Properties***

$t_f := 1.25in$	*Bottom Flange Thickness*
$D_w := 60in$	*Web Depth*
$t_w := 0.75in$	*Web Thickness*
$S_{gird} := 7.5ft$	*Girder Spacing*
$t_{pl} := 1.0in$	*Connection Plate Thickness*
$b_{pl} := 9.5in$	*Connection Plate Width*
$skew := 45deg$	*Bearing to Girder Skew Angle*
$haunch := 4.5in$	*Clear distance between bottom of deck and bottom of top flange*

- ***Bolt Properties/Geometries***

$d_{bolt} := 0.875in$ *Diameter of H.S. Bolt for X-Frame Connection*

$d_{h_tens} := 0.9375in$ *Design Bolt Hole for Tension, Block Rupture [AASHTO 6.8.3 or 6.13.4]*

$d_{h_stndrd} := d_{bolt} + 0.0625in$ $\quad d_{h_stndrd} = \frac{15}{16}in$ *Standard Bolt Hole Diameter [6.13.2.4]*

$S_{g.end} := 1.50in$ $\quad S_{pl.end} := 1.5in$ $\quad S_{g.row} := 3.0in$ *Bolt Hole Geometry, See Diagram*

$S_{t2} := 3.0in$ $\quad S_{t.end} := 1.50in$ $\quad S_{t.pl} := 1.5in$ $\quad S_{t1} := 3.0in$ *Top connection dimensions, see diagram*

$A_b := \pi \cdot (0.5 \cdot d_{bolt})^2$ $\quad A_b = 0.601 \cdot in^2$ *Area of Single Bolt*

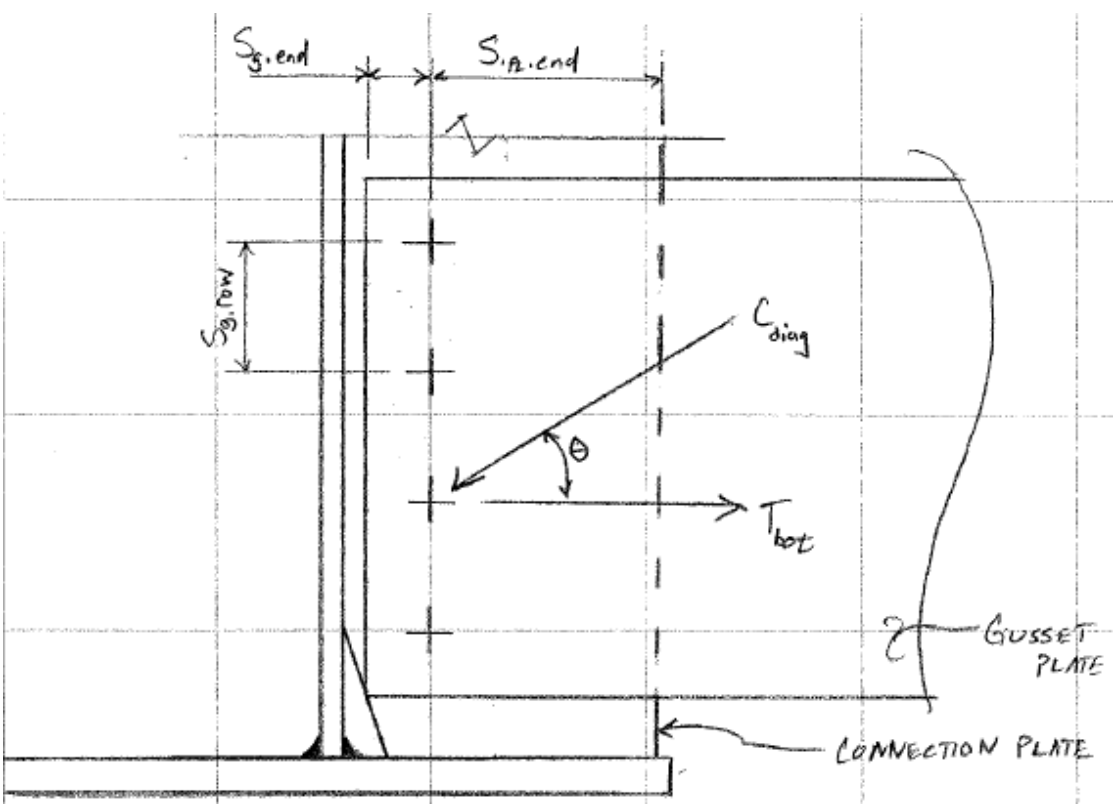
$P_t := 39kip$ *Minimum Bolt Tension (AASHTO Table 6.13.2.8-1)*

$K_h := 1.0$ *Hole Size Factor (AASHTO Table 6.13.2.8-2)*

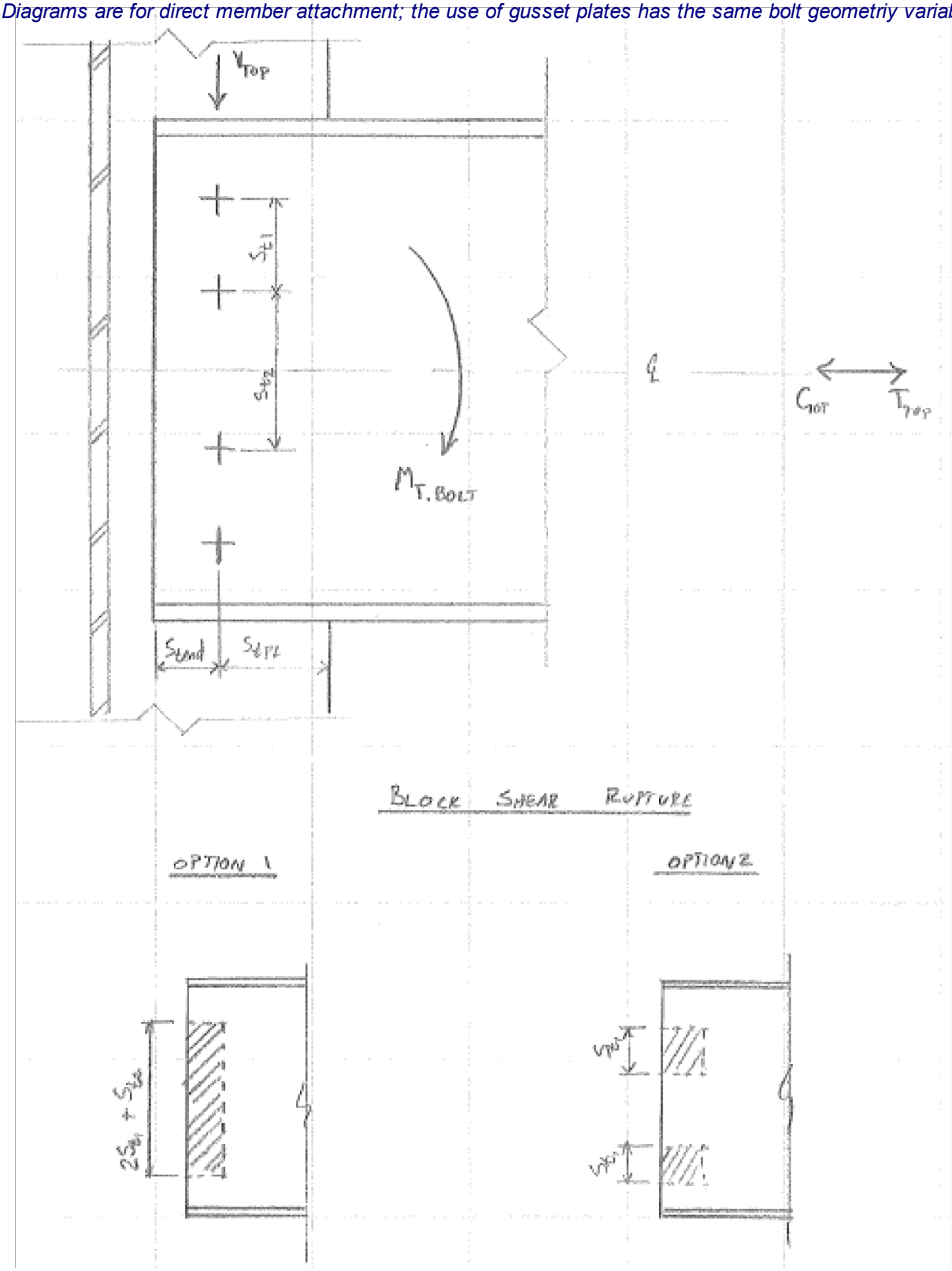
$K_s := 0.50$ *Surface Condition Factor (AASHTO Table 6.13.2.8-3) - Class B Surface*

$N_s := 1$ *Number of Slip Planes*

$N_{bolts} := 4$ *Number of Bolts per Connection*



Diagrams are for direct member attachment; the use of gusset plates has the same bolt geometriy variables.



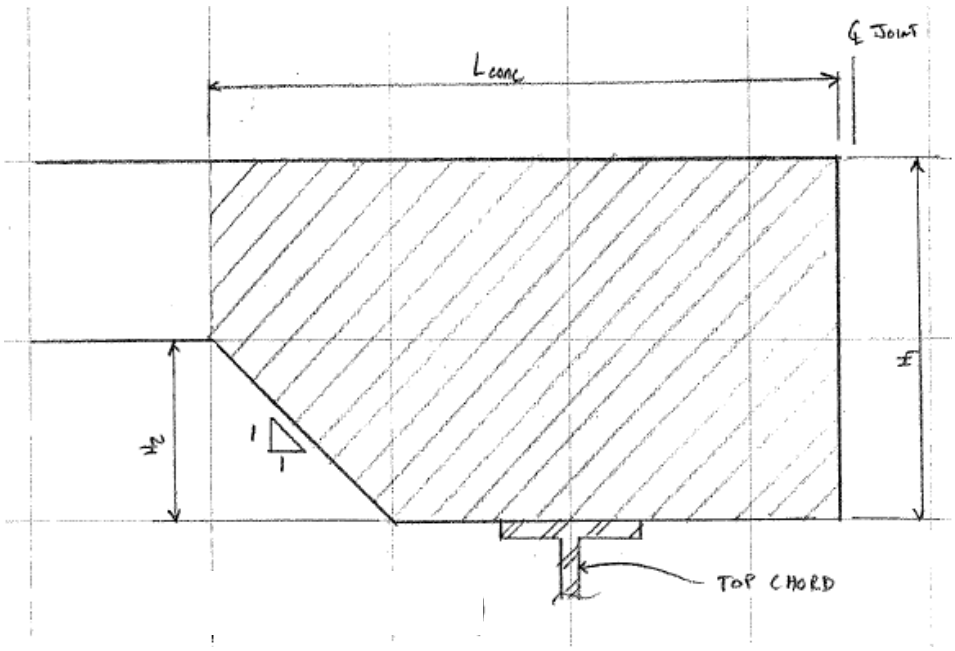
- ***Concrete Edge Beam Geometry***

$H_1 := 9.0in + haunch + 4in = 17.50\,in$ — *Total Depth of Beam (deck+haunch+offset below flange)*

$H_2 := H_1 - 9.0in = 8.5\,in$ — *Depth of Beam Beyond Deck Thickness*

$L_{conc} := 2 \cdot 13in + 0in = 26\,in$ — *Width of Beam Supported by Top Chord (No 1:1 Bevel included)*

$A_{conc} := H_1 \cdot L_{conc}$ $\quad A_{conc} = 3.16 \cdot ft^2$ — *Area of Beam*



- ***Cross Frame Geometry***

Approximate for calculation purposes only

$H_{xf} := Round(D_w - 7.5in - 5.5in, 1in)$ $\quad H_{xf} = 47\,in$ — *Height Between CL Top & Bot Chord*

$W_{xf} := \frac{S_{gird}}{\cos(skew)} - \frac{t_w}{\cos(skew)} - \frac{(b_{pl})}{\cos(skew)} - 4in$ $\quad W_{xf} = 108.78\,in$ — *Horizontal Dist Between Center of connection of top/bottom chord*

$\theta := atan\left[\frac{H_{xf}}{0.5W_{xf} + (b_{pl})}\right]$ $\quad \theta = 36.34 \cdot deg$ — *Angle Between Members*

$H_{dg} := 31in$ $\quad H_{dg} = 31\,in$ — *Vertical Height of Diagonals*

$L_h := W_{xf}$ $\quad L_h = 108.78\,in$ — *Length of Top and Bot Chords*

$L_{dg} := \sqrt{H_{dg}^2 + (0.5W_{xf})^2}$ $\quad L_{dg} = 62.61\,in$ — *Length of Diagonals*

$t_{gus} := 0.75in$ — *Thickness of Gusset Plates*

$h_{gus.b} := 18in$ — *Height of Bottom Gusset Plates*

$h_{gus.t} := 12in$ — *Height of Top Gusset Plates*

- *L4x4x1/2 Properties (AISC Table 1-7)*

$A_{ang} := 3.75in^2$

$b_{f.ang} := 4in$

$t_{f.ang} := 0.5in$

$r_{xy.ang} := 1.21in$

$r_{z.ang} := 0.776in$

$J_{ang} := 0.322in^4$

$I_{y.ang} := 5.52in^4$

$S_{x.ang} := 1.96in^3$

$y_{o.ang} := 1.18in$

- *MC8x22.8 Properties (AISC Table 1-6)*

$A_i := 6.7in^2$

$d_i := 8.0in$

$t_{w.i} := 0.427in$

$k_i := 1.1875in$

$b_{f.i} := 3.50in$

$t_{f.i} := 0.525in$

$r_{y.i} := 1.02in$

$r_{x.i} := 3.09in$

$y_o := 1.04in$

$I_{x.i} := 63.8in^4$

$I_{y.i} := 7.01in^4$

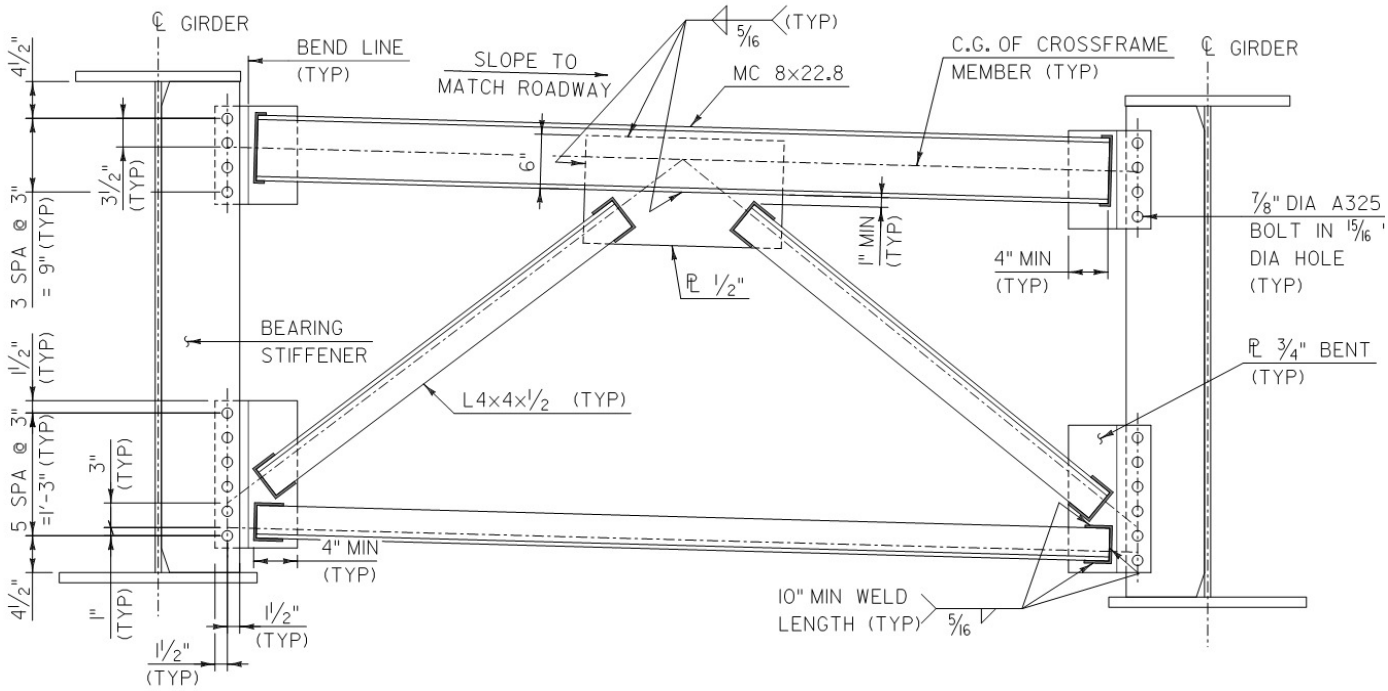
$Z_{x.i} := 19.1in^3$

$Z_{y.i} := 5.37in^3$

$S_{x.i} := 15.8in^3$

$J_i := 0.572in^4$

$C_{w.i} := 75.2in^6$



ABUTMENT 2 END CROSSFRAME DETAIL

SCALE: 1 1/2" = 1'-0"

Wind Forces:

Computation of Wind Forces in CF members:

Wind pressure $p_w := 50.0psf$ *AASHTO LRFD Table 3.8.1.2.1-1*

Tributary length for abutment $L_{trib} := \frac{172ft}{2} = 86 \cdot ft$

Superstructure height: $H := 7in + 9in + haunch + 60in + 1.75in = 6.85 \cdot ft$

$H_{misc} := (8 + 4)in = 12\,in$ *from horizontal rails (approx.)*

$H_{tot} := H + H_{misc} = 7.85 \cdot ft$ *exposed height for wind*

Wind load on bottom flange at abutment CF

$F_w := \frac{-p_w \cdot L_{trib} \cdot H_{tot}}{2} = -16.89 \cdot kip$ *Compression load, the other half goes thru deck slab*

$F_{diag} := F_w \cdot \left(\frac{\cos(\theta)}{1 + \cos(\theta)}\right) \cdot \frac{1}{\cos(\theta)} = -9.35\,kip$ *Axial componenet of portion of wind load carried by diagonal*

$F_{bot.chord} := F_w \cdot \frac{1}{1 + \cos(\theta)} = -9.35\,kip$ *Portion of wind load carried by bottom chord*

$F_{top.chord} := 0kip$ *No additional loads in top chord due to balancing of diagonals.*

MDX Member Forces:

- *Maximum Factored Loads Among All End Bearing Cross Frames, Taken from MDX Output and/or Excel*

$T_{top} := 26.08kip$ *Tension in Top Chord*

$C_{top} := -23.6kip$ *Compression in Top Chord*

$T_{bot.mdx} := 2.39kip$ *Tension in Bottom Chord*

$C_{bot.mdx} := -9.44kip$ *Compression in Bottom Chord*

$T_{diag.mdx} := 33.58kip$ *Tension in Diagonal Chord*

$C_{diag.mdx} := -33.58kip$ *Compression in Diagonal Chord*

- *Fatigue Stress Range*

$\sigma_{top.fat.mdx} := 1.77ksi$ *Fatigue Stress Range in Top Chord*

$\sigma_{bot.fat.mdx} := 1.58ksi$ *Fatigue Stress Range in Bottom Chord*

$\sigma_{diag.fat.mdx} := 2.61ksi$ *Fatigue Stress Range in Diagonal Chord*

- *Service II Forces*

$F_{top.mdx} := 19.34kip$ *Absolute Max Force in Top Chord*

$F_{bot.mdx} := 7.01kip$ *Absolute Max Force in Bottom Chord*

$F_{diag.mdx} := 24.92kip$ *Absolute Max Force in Diagonal Chord*

Fatigue Thresholds:

Considering low traffic count, use Fatigue II load combination (finite fatigue life)

$n := 1.0$ *Cycles per truck passage*

$ADTT := 330$ *Single lane traffic count, extrapolated to 50 year mark*

$N := 365 \cdot 100 \cdot n \cdot ADTT = 1.2 \times 10^7$ *Total cycles, assuming 100 year life*

$A_B := 120 \cdot 10^8 (ksi)^3$ *Fatigue Detail Category Constant, Category B (AASHTO Table 6.6.1.2.5-1)*

$A_E := 11 \cdot 10^8 (ksi)^3$ *Fatigue Detail Category Constant, Category E (AASHTO Table 6.6.1.2.5-1)*

$$F_{n.B} := \left(\frac{A_B}{N}\right)^{\left(\frac{1}{3}\right)} = 9.99 \cdot ksi$$

Allowable Fatigue Stress Rabge for Category B Details (AASHTO 6.6.1.2.5-2)

$$F_{n.E} := \left(\frac{A_E}{N}\right)^{\left(\frac{1}{3}\right)} = 4.5 \cdot ksi$$

Allowable Fatigue Stress Rabge for Category E Details (AASHTO 6.6.1.2.5-2)

Geometry Checks:

$$Check_1 := if\left[H_{xf} + \frac{d_i}{2} + \frac{b_{f.ang}}{2} \geq 0.75\left(D_w + 2t_f\right), \text{"Cross Frame Depth OK"}, \text{"Cross Frame Depth Needs to Be Increased"}\right]$$

$Check_1 = \text{"Cross Frame Depth OK"}$ *[AASHTO 6.7.4.2]*

- *Slenderness Checks*

$K := 0.75$

$K_{ang} := 1.0$

Effective Length Factors (4.6.2.5)

$KL_R_{all} := 140$ *Allowable Slenderness [6.9.3]*

$K_{ang} \cdot L_h \div \min\left(r_{xy.ang}, r_{z.ang}\right) = 140.18$ *Bottom Chord Slenderness Ratio [6.9.3]*

$K_{ang} \cdot L_{dg} \div \min\left(r_{xy.ang}, r_{z.ang}\right) = 80.68$ *Diag Member Slenderness Ratio [6.9.3]*

$K \cdot \frac{L_h}{2} \div \min\left(r_{x.i}, r_{y.i}\right) = 39.99$ *Top Chord Slenderness Ratio [6.9.3]*

$$Check_2 := if\left(\frac{K_{ang} \cdot L_h}{\min\left(r_{xy.ang}, r_{z.ang}\right)} < KL_R_{all}, \text{"Bottom Chord Slenderness OK"}, \text{"Bottom Chord Slenderness No Good"}\right)$$

$Check_2 = \text{"Bottom Chord Slenderness No Good"}$ *Say OK - consistent with original design and only over the limit by 0.1%*

$$Check_3 := if\left(\frac{K \cdot L_h}{2\min\left(r_{x.i}, r_{y.i}\right)} < KL_R_{all}, \text{"Top Chord Slenderness OK"}, \text{"Top Chord Slenderness No Good"}\right)$$

$Check_3 = \text{"Top Chord Slenderness OK"}$

$$Check_4 := if\left(\frac{K_{ang} \cdot L_{dg}}{\min\left(r_{xy.ang}, r_{z.ang}\right)} < KL_R_{all}, \text{"Diagonal Chord Slenderness OK"}, \text{"Diag Chord Slenderness No Good"}\right)$$

$Check_4 = \text{"Diagonal Chord Slenderness OK"}$

$$Check_5 := if\left(\frac{b_{f.ang}}{t_{f.ang}} < 0.45\sqrt{\frac{E_s}{F_y}}, \text{"Angle Plate Slenderness OK"}, \text{"Angle Leg is Too Slender, No Good"}\right)$$

$Check_5 = \text{"Angle Plate Slenderness OK"}$ *[AASHTO 6.9.4.2-1]*

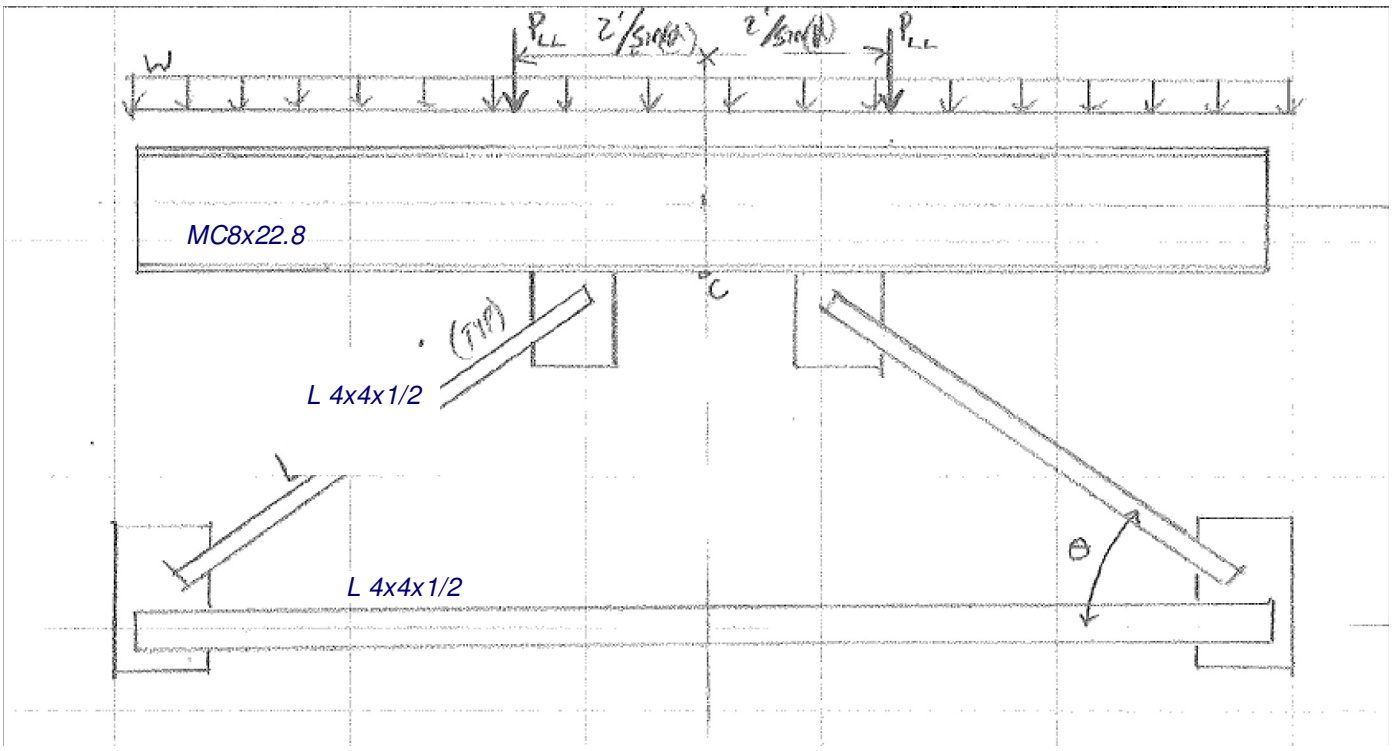
Bending Loads:

For force distribution, use deflection compatability and superposition to find equivalent center deflection due to gravity loads and equivalent stiffness of diagonal members pushing vertically.

$w_{conc} := 150pcf \cdot A_{conc}$ $\quad w_{conc} = 473.96 \cdot plf$ *Concrete Edge Beam Dist Load*

$w_{self} := A_i \cdot 490pcf$ $\quad w_{self} = 22.8 \cdot plf$ *Self weight of Top Chord*

$P_{LL} := 16kip$ *Live Load Wheel Load*



- ***Strength I***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.2 \cdot 1.33 \cdot 1.75 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} + \frac{5\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$a := \mathrm{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right)$ $\delta_{str1} := a_0 \cdot in = 0.023 \cdot in$ $P_{str.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 39.36 kip$ *Midspan deflection and axial force in diagonals*

$M_{Str1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{str1}}{L_h^2} = 3.54 \cdot kip \cdot ft$ *Moment in Top Chord*

$V_{Str.1} := 0.5\left[1.2 \cdot 1.33 \cdot 1.75 P_{LL} + \left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 1.84 kip$ *Shear at ends of Chord*

- *Wheel loads from two passing trucks*

$b := \left(\frac{L_h}{2} - \frac{2ft}{\cos(skew)}\right)$ *Distance from beam support to wheel load application*

guess $\delta_2 := 0.03in$ $P_{s2} := 6kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.33 \cdot 1.75 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] + \frac{5\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$a := \mathrm{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right)$ $\delta_{str2} := a_0 \cdot in = 0.02 \cdot in$ $P_{str.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 35.55 kip$ *Midspan deflection and axial force in diagonals*

$M_{Str2.pos} := 1.33 \cdot 1.75 P_{LL} \cdot (b) + \frac{\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot (b)}{2} \cdot \left[L_h - (b)\right] - \frac{a_1 \cdot kip \cdot (b)}{2} = 31.46 \cdot kip \cdot ft$ *Max Pos Moment*

$M_{Str2.neg} := 1.33 \cdot 1.75 P_{LL} \cdot (b) + \frac{\left(1.25 w_{conc} + 1.25 w_{self}\right)}{8} \cdot \left(L_h^2\right) - \frac{a_1 \cdot kip \cdot \left(L_h\right)}{4} = -25.64 \cdot kip \cdot ft$ *Max Neg Moment*

$V_{Str.2} := 0.5\left[2 \cdot 1.33 \cdot 1.75 P_{LL} + \left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 18.99 kip$ *Shear at ends of Chord*

- *Controlling Forces*

$M_{u.Str1} := \max\left(M_{Str1}, M_{Str2.pos}, \left|M_{Str2.neg}\right|\right) = 31.46 \cdot kip \cdot ft$ *Maximum Moment in Top Chord*

$P_{diag.str} := \max\left(P_{str.1}, P_{str.2}\right) = 39.36 kip$ *Maximum Axial load in diagonals*

$V_{u.Str} := \max\left(V_{Str.1}, V_{Str.2}\right) = 18.99 kip$ *Maximum Shear at ends of Top Chord*

- ***Service II***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.2 \cdot 1.33 \cdot 1.30 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} + \frac{5\left(w_{conc} + w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$

Midspan deflection in Top Chord

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$

Deflection in diagonals (treated as a spring)

$$a := \text{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right) \quad \delta_{ser1} := a_0 \cdot in = 0.017 \cdot in \quad P_{ser.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 29.4\,kip$$

Midspan deflection and axial force in diagonals

$$M_{Ser1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{ser1}}{L_h^2} = 2.65 \cdot kip \cdot ft$$

Moment in Top Chord

$$V_{Ser.1} := 0.5\left[1.2 \cdot 1.33 \cdot 1.30 P_{LL} + \left(w_{conc} + w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 1.43\,kip$$

Shear at ends of Chord

- *Wheel loads from two passing trucks*

$$b := \left(\frac{L_h}{2} - \frac{2ft}{\cos(skew)}\right)$$

Distance from beam support to wheel load application

guess $\delta_2 := 0.03in$ $P_{s2} := 6kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.33 \cdot 1.30 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] + \frac{5\left(w_{conc} + w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$

Midspan deflection in Top Chord

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$

Deflection in diagonals (treated as a spring)

$$a := \text{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right) \quad \delta_{ser2} := a_0 \cdot in = 0.015 \cdot in \quad P_{ser.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 26.57\,kip$$

Midspan deflection and axial force in diagonals

$$M_{Ser2.pos} := 1.33 \cdot 1.30 P_{LL} \cdot (b) + \frac{\left(w_{conc} + w_{self}\right) \cdot (b)}{2} \cdot \left[L_h - (b)\right] - \frac{a_1 \cdot kip \cdot (b)}{2} = 23.43 \cdot kip \cdot ft$$

Max Pos Moment

$$M_{Ser2.neg} := 1.33 \cdot 1.30 P_{LL} \cdot (b) + \frac{\left(w_{conc} + w_{self}\right)}{8} \cdot \left(L_h^2\right) - \frac{a_1 \cdot kip \cdot \left(L_h\right)}{4} = -19.12 \cdot kip \cdot ft$$

Max Neg Moment

$$V_{Ser.2} := 0.5\left[2 \cdot 1.33 \cdot 1.30 P_{LL} + \left(w_{conc} + w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 14.17\,kip$$

Shear at ends of Chord

- *Controlling Forces*

$$M_{Serv2} := \max\left(M_{Ser1}, M_{Ser2.pos}, \left|M_{Ser2.neg}\right|\right) = 23.43 \cdot kip \cdot ft$$

Maximum Moment in Top Chord

$$P_{diag.ser2} := \max\left(P_{ser.1}, P_{ser.2}\right) = 29.4\,kip$$

Maximum Axial load in diagonals

$$V_{u.Ser2} := \max\left(V_{Ser.1}, V_{Ser.2}\right) = 14.17\,kip$$

Maximum Shear at ends of Top Chord

- ***Fatigue***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.15 \cdot 0.75 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$$a := \text{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right) \quad \delta_{fat1} := a_0 \cdot in = 0.006 \cdot in \quad P_{fat.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 11.27\,kip$$ *Midspan deflection and axial force in diagonals*

$$M_{fat1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{fat1}}{L_h^2} = 1.01 \cdot kip \cdot ft$$ *Moment in Top Chord*

$$V_{fat.1} := 0.5\left(1.15 \cdot 1.50 P_{LL} - a_1 \cdot kip\right) = 7.12\,kip$$ *Shear at ends of Chord*

- *Wheel loads from single passing trucks, centered over bay*

$$b := \left(\frac{L_h}{2} - \frac{3ft}{\cos(skew)}\right)$$ *Distance from beam support to wheel load application*

guess $\delta_2 := 0.003in$ $P_{s2} := 5kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.15 \cdot 0.75 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$$a := \text{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right) \quad \delta_{fat2} := a_0 \cdot in = 0.001 \cdot in \quad P_{fat.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 2.16\,kip$$ *Midspan deflection and axial force in diagonals*

$$M_{fat2.pos} := 1.15 \cdot 0.75 P_{LL} \cdot (b) - \frac{a_1 \cdot kip \cdot (b)}{2} = 3.63 \cdot kip \cdot ft$$ *Max Pos Moment*

$$M_{fat2.neg} := 1.15 \cdot 0.75 P_{LL} \cdot (b) - \frac{a_1 \cdot kip \cdot (L_h)}{4} = -1.8 \cdot kip \cdot ft$$ *Max Neg Moment*

$$V_{fat.2} := 0.5\left(2 \cdot 1.15 \cdot 0.75 P_{LL} - a_1 \cdot kip\right) = 12.52\,kip$$ *Shear at ends of Chord*

- *Controlling Forces*

$$M_{Fat} := \max\left(M_{fat1}, M_{fat2.pos}, \left|M_{fat2.neg}\right|\right) = 3.63 \cdot kip \cdot ft$$ *Maximum Moment in Top Chord*

$$P_{diag.fat} := \max\left(P_{fat.1}, P_{fat.2}\right) = 11.27\,kip$$ *Maximum Axial load in diagonals*

$$V_{u.fat} := \max\left(V_{fat.1}, V_{fat.2}\right) = 12.52\,kip$$ *Maximum Shear at ends of Top Chord*

Top Chord Design Checks:

- ***Compression/Bending***

  - *Nominal Design Load*

$\phi_c := 0.95$ *Resistance Factor [6.5.4.2]*

$$Q := \begin{cases} 1 & \text{if } \frac{b_{f.i}}{t_{f.i}} \le 0.56\cdot\sqrt{\frac{E_s}{F_y}} \wedge \frac{d_i - 2\cdot k_i}{t_{w.i}} \le 1.49\cdot\sqrt{\frac{E_s}{F_y}} \\ \text{"Slender"} & \text{otherwise} \end{cases} = 1$$

Slender Element Reduction Factor [AASHTO 6.9.4.2.1-1]

$P_o := Q\cdot F_y\cdot A_i = 335\cdot kip$ *Equivalent nominal yield resisitance [AASHTO 6.9.4.1.1]*

Flexural Buckling

$$P_{e.FB} := \frac{\pi^2\cdot E_s}{\left(\frac{K\cdot L_h \div 2}{\min(r_{x.i}, r_{y.i})}\right)^2}\cdot A_i = 1198.9\cdot kip$$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

Flexural Torsional Buckling

$$P_{ex} := \frac{\pi^2\cdot E_s}{\left(\frac{K\cdot L_h \div 2}{r_{x.i}}\right)^2}\cdot A_i = 1.1\times 10^4\,kip$$

AASHTO 6.9.4.1.3-4

$$r_o := \sqrt{y_o^2 + \frac{I_{x.i} + I_{y.i}}{A_i}} = 3.41\,in$$

Polar Radius of Inertia (AASHTO 6.9.4.1.3-6)

$$P_{ez} := \left[\frac{\pi^2\cdot E_s\cdot C_{w.i}}{(K\cdot L_h \div 2)^2} + G\cdot J_i\right]\cdot\frac{1}{r_o^2} = 1110.72\,kip$$

AASHTO 6.9.4.1.3-5

$$H_i := 1 - \frac{y_o^2}{r_o^2} = 0.91$$

AASHTO 6.9.4.1.3-3

$$P_{e.FTB} := \frac{P_{ex} + P_{ez}}{2H_i}\cdot\left[1 - \sqrt{1 - \frac{4P_{ex}\cdot P_{ez}\cdot H_i}{(P_{ex} + P_{ez})^2}}\right] = 1099.39\,kip$$

Elastic Buckling base on FTB, (AASHTO 6.9.4.1.3-2)

$$P_{e.i} := \min(P_{e.FB}, P_{e.FTB}) = 1099.39\,kip$$

$$P_{n.i} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.i}}}\cdot P_o\right) & \text{if } \frac{P_{e.i}}{P_o} \ge 0.44 \\ (0.877\cdot P_{e.i}) & \text{otherwise} \end{cases} = 294.89\cdot kip$$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c \cdot P_{n.i} = 280.14\,kip$ *Factored Compressive Resistance*

$M_{uy_Str} := \left|C_{top}\right| \cdot (y_o) = 2.05 \cdot kip \cdot ft$ *Bending about the y-axis due to eccentric compression load*

$M_{ry} := F_y \cdot Z_{y.i} = 22.37 \cdot kip \cdot ft$ *Moment capacity about the y-axis, fully braced on top, mid point brace on bottom, therefore assume full capacity can be reached.*

$M_r := F_y \cdot Z_{x.i}$ $M_r = 79.58 \cdot kip \cdot ft$ *Resistive moment, noncomposite, fully braced compresion flange.*

$$Bend_Ratio := \begin{cases} \dfrac{\left|C_{top}\right|}{2(\phi_c \cdot P_{n.i})} + \dfrac{M_{u.Str1}}{M_r} + \dfrac{M_{uy_Str}}{M_{ry}} & \text{if } \dfrac{\left|C_{top}\right|}{\phi_c \cdot P_{n.i}} < 0.2 \\ \dfrac{\left|C_{top}\right|}{\phi_c \cdot P_{n.i}} + \dfrac{8}{9}\left(\dfrac{M_{u.Str1}}{M_r} + \dfrac{M_{uy_Str}}{M_{ry}}\right) & \text{otherwise} \end{cases}$$

[6.9.2.2-1]

[6.9.2.2-2]

$Bend_Ratio = 0.53$

$Check_6 := \text{if}(Bend_Ratio < 1.0, \text{"Top Chord OK in Comp/Bend"}, \text{"Top Chord No Good in Comp/Bend"})$

$Check_6 = \text{"Top Chord OK in Comp/Bend"}$

- ***Tension (w/o Bending)***
  - *Yielding and Fracture*

$\phi_y := 0.95$ *Resistance Factor for Yielding [6.5.4.2]*

$\phi_u := 0.80$ *Resistance Factor for Fracture [6.5.4.2]*

$U := 0.75$ *Reduction Factor [6.8.2.2]*

$A_{n.i} := A_i = 6.7\,in^2$ *Net Area of Member in Tension*

$\phi P_{ny} := \phi_y \cdot F_y \cdot A_i = 318.25\,kip$ *Factored Tensile Resistance in Yielding [6.8.2.1-1]*

$\phi P_{nu} := \phi_u \cdot F_u \cdot A_{n.i} \cdot U = 261.3\,kip$ *Factored Tensile Resistance in Fracture [6.8.2.1-2]*

$\phi P_{r.tens} := \min(\phi P_{ny}, \phi P_{nu}) = 261.3\,kip$ *Factored Tensile Resistance [6.8.2.1]*

$Check_7 := if(\phi P_{r.tens} \geq |T_{top}|, \text{"Top Chord OK in Tension"}, \text{"Top Chord No Good in Tension"})$

$Check_7 = \text{"Top Chord OK in Tension"}$

- ***Determine Weld Length Along Member***

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld*

$F_{n.B} = 9.99 \cdot ksi$ *Fatigue Threshold in Weld Matal [AASHTO 6.6.1.2.5-3, Category B]*

$L_{fat.req.w} := \dfrac{\sigma_{top.fat.mdx} \cdot A_i}{F_{n.B} \cdot 0.221in} = 5.37\,in$ *Required Length of Weld, Based on Fatigue inthe weld metal and assuming a 5/16" weld size*

$L_{Str.req} := \dfrac{\max(|C_{top}|, T_{top}, V_{u.Str})}{\phi_{e2} \cdot 0.60 F_{exx} \cdot 0.221in} = 3.51\,in$ *Required Length of Weld, Based on Strength I and assuming a 5/16" weld size*

$Weld_{top} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$ $Weld_{top} = 10\,in$ *Ten inches included to match VTrans plans*

- ***Check Fatigue in Member***

Away from welds

$\sigma_{top.fat} := \sigma_{top.fat.mdx} + \dfrac{M_{Fat}}{S_{x.i}} = 4.53 \cdot ksi$ *Fatigue Stress range in Member, away from connection*

$\sigma_{top.fat.mdx} = 1.77 \cdot ksi$ *Fatigue Stress Range in member near welded connections*

$Check_8 := if(\sigma_{top.fat} < F_{n.B} \wedge \sigma_{top.fat.mdx} < F_{n.E}, \text{"Member OK in Fatigue"}, \text{"Member No Good in Fatigue"})$

$Check_8 = \text{"Member OK in Fatigue"}$

- ***Check Bent Plate Capacity***

Check plate thickness to prevent prying action in accordance with AISC 13th edition, Chapter 9.

$$T := \frac{\max\left(\left|C_{top}\right|, T_{top}\right)\cdot\sin(\theta)}{4} = 3.86\,kip$$

Equivalent prying force, per bolt

$$b' := S_{pl.end} + 1\,in - \frac{d_{bolt}}{2} = 2.06\,in$$

Distance from bend in plate to edge of bolt hole

$$t_{min} := \sqrt{\frac{4.44\cdot T\cdot b'}{S_{t1}\cdot F_u}} = 0.43\,in$$

Minimum plate thickness required to avoid additional prying force in bolt

$$Check_{pry} := if\left(t_{min} < t_{gus}, \text{"No Bolt Prying will occur"}, \text{"Bolt prying will occur, Thicken Gusset Plate"}\right)$$

$$\boxed{Check_{pry} = \text{"No Bolt Prying will occur"}}$$

Check Horizontal Bending Stress in Plate

$$M_{pl} := 4\cdot T\cdot\left(b' + \frac{d_{bolt}}{2} + 0.5\cdot S_{g.end}\right) = 50.23\cdot kip\cdot in$$

Bending Moment in Gusset plate - assume point of fixity is half way between bolts and tip of gusset in compression

$$M_r := F_y\cdot\frac{h_{gus.t}\cdot t_{gus}^{2}}{6} = 56.25\cdot kip\cdot in$$

Moment capacity of plate, assuming section can reach first yield in section under consideration

$$Check_{pl} := if\left(M_r > M_{pl}, \text{"Bending in Gusset Plate Tip OK"}, \text{"Bending in Gusset Plate Tip No Good - Thicken Plate"}\right)$$

$$\boxed{Check_{pl} = \text{"Bending in Gusset Plate Tip OK"}}$$

Check Vertical Bending in Plate

$V_{top.str} := V_{u.Str} = 18.99\,kip$ *Vertical shear*

$L_b := \frac{(6in + 1in + S_{g.end})}{\cos(skew)^2} = 17\,in$ *Cantilever length and unbraced length of gusset from end of gusset to bolts, increased by skew to help account for out-of-plane effects*

$M_{pl.v} := V_{top.str} \cdot (L_b) = 322.8 \cdot kip \cdot in$ *Vertical Bending moment in plate due to gravity loads*

$C_b := 1.0$ *Lateral Torsional Buckling coeeficietn (AISC F1-1) - conservatively take as 1.0*

Moment Capacity per AISC Specifications Section F11:

$$M_p := F_y \cdot 1.5 \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6} = 1.35 \times 10^3 \cdot kip \cdot in$$

$$M_n := \begin{cases} M_p & \text{if } \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} < \frac{0.08E_s}{F_y} \\ \min\left[M_p, C_b \cdot \left[1.52 - 0.274 \cdot \left(\frac{L_b \cdot h_{gus.t}}{t_{gus}^2}\right) \cdot \frac{F_y}{E_s}\right] \cdot \left(F_y \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6}\right)\right] & \text{if } \left(\frac{0.08E_s}{F_y} < \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} \wedge \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} < \frac{1.9E_s}{F_y}\right) \\ \min\left(M_p, \frac{1.9 \cdot E_s \cdot C_b}{\frac{L_b \cdot h_{gus.t}}{t_{gus}^2}} \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6}\right) & \text{otherwise} \end{cases}$$

$M_n = 1.21 \times 10^3 \cdot kip \cdot in$ *Moment Capacity of plate (reduction factor is 1.0)*

$Check_{pl.v} := \text{if}(M_n > M_{pl.v}, \text{"Vertical Bending in Plate OK"}, \text{"Vertical Bending No Good, Thicken Plate"})$

$Check_{pl.v} = \text{"Vertical Bending in Plate OK"}$

- ***Connection Capacity Checks***

- *Block Shear Rupture*

$\phi_{bs} := 0.80$ *Resistance Factor for Block Shear*

$t := \min(t_{gus}, t_{pl}) = 0.75\,in$ *Minimum plate thickness for rupture checks*

- *Member Capacity - Option 1*

$A_{vg} := 2 \cdot S_{t.end} \cdot t = 2.25\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := 2 \cdot (S_{t.end} - 0.5d_{h_tens}) \cdot t = 1.55\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (2S_{t1} + S_{t2}) \cdot t = 6.75\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := (2S_{t1} + S_{t2} - 3d_{h_tens}) \cdot t = 4.64\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.i1} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 287.97\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

- *Member Capacity - Option 2*

$A_{vg} := 4 \cdot S_{t.end} \cdot t = 4.5\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := 4 \cdot (S_{t.end} - 0.5d_{h_tens}) \cdot t = 3.09\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (2S_{t1}) \cdot t = 4.5\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := (2S_{t1} - 2d_{h_tens}) \cdot t = 3.09\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.i2} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 254.18\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

$Check := if(\min(R_{r.i1}, R_{r.i2}) > T_{top}, \text{"Horiz Block Rupture OK"}, \text{"Horiz Block Rupture No Good"})$

Check = "Horiz Block Rupture OK"

- *Vertical Block Rupture in Gusset*

$V_{top.str} = 18.99\,kip$ *Vertical shear*

$A_{vg} := (S_{t.end} + 2 \cdot S_{t1} + S_{t2}) \cdot t = 7.87\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := A_{vg} - 3.5d_{h_tens} \cdot t = 5.41\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (S_{t.end}) \cdot t = 1.12\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := A_{tg} - 0.5d_{h_tens} \cdot t = 0.77\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.v} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 203.51\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

$Check := if(R_{r.v} > V_{top.str}, \text{"Vertical Block Rupture OK"}, \text{"Veritcal Block Rupture No Good"})$

Check = "Vertical Block Rupture OK"

- ***Bolt Forces***

$$I_p := 2\left[\left(\frac{S_{t2}}{2}\right)^2 + \left(\frac{S_{t2}}{2} + S_{t1}\right)^2\right] \qquad I_p = 45\,in^2$$

Bolt Pattern Inertia

$$y_{max} := S_{t1} + \frac{S_{t2}}{2} \qquad y_{max} = 4.5\,in$$

Distance from Bolt Group CG to outer Bolt

- *Strength I*

$$V_{top.str} = 18.99\,kip$$

Shear Acting on Bolts due to LL+IM

$$M_{t.bolt.str} := V_{top.str}\cdot\left(5in + S_{t.end}\right) = 10.29\cdot kip\cdot ft$$

Moment Acting on Bolts due to eccentricity

$$F_{m.str} := \frac{M_{t.bolt.str}\cdot y_{max}}{I_p} = 12.34\,kip$$

Force on Outer Bolt Due to moment

$$V_{bolt.str} := \sqrt{\left(F_{m.str} + \frac{\max\left(\left|C_{top}\right|, T_{top}\right)}{N_{bolts}}\right)^2 + \left(\frac{V_{top.str}}{N_{bolts}}\right)^2} = 19.45\,kip$$

Ultimate Shear Force per Bolt, Strength I

- *Service II*

$$V_{top.serv} := V_{u.Ser2} = 14.17\,kip$$

Shear Acting on Bolts due to LL+IM

$$M_{t.bolt.serv} := V_{top.serv}\cdot\left(5in + S_{t.end}\right) = 7.68\cdot kip\cdot ft$$

Moment Acting on Bolts due to eccentricity

$$F_{m.serv} := \frac{M_{t.bolt.serv}\cdot y_{max}}{I_p} = 9.21\,kip$$

Force on Outer Bolt Due to moment

$$V_{bolt.serv} := \sqrt{\left(F_{m.serv} + \frac{F_{top.mdx}}{N_{bolts}}\right)^2 + \left(\frac{V_{top.serv}}{N_{bolts}}\right)^2} = 14.48\,kip$$

Ultimate Shear Force per Bolt, Service II

- *Fatigue*

$V_{top.fat} := V_{u.fat} = 12.52\,kip$ *Shear Acting on Bolts due to LL+IM*

$M_{t.bolt.fat} := V_{top.fat} \cdot (S_{t.end} + 5in) = 6.78 \cdot kip \cdot ft$ *Moment Acting on Bolts due to eccentricity*

$$\sigma_{top.fat} := \frac{M_{t.bolt.fat} \cdot (S_{t1} + 0.5S_{t2})}{\frac{t_{gus} \cdot h_{gus.t}^{3}}{12}} + \sigma_{top.fat.mdx} = 5.16 \cdot ksi$$

Fatigue Stress Acting in plate at Extreme Bolt Location

- ***Bolt Bearing on Material***

$t_{mat} := \min(t_{pl}, t_{w.i}) = 0.43\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min[(S_{t1} - d_{h_stndrd}), (S_{t2} - d_{h_stndrd}), (S_{t.end} - 0.5d_{h_stndrd})] = 1.03\,in$

Minimum of Clear Distance Between Holes Or Edge of Material in the Direction of Loading [6.13.2.9]

$$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases} = 34.35\,kip$$

Nominal Resistance in Bearing, Per Bolt

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 27.48\,kip$ *Resistance of Single Bolt in Bearing*

$Check_9 := \text{if}(|V_{bolt.str}| \le \phi_{bb} \cdot R_n, \text{"Top Chord Bolt Bearing OK"}, \text{"Top Chord Bolt Bearing No Good"})$

$Check_9 = \text{"Top Chord Bolt Bearing OK"}$

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [AASHTO 6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Factored Resistance Per Bolt*

$Check_{10} := \text{if}(V_{bolt.str} \le \phi_s \cdot F_{v_bolt}, \text{"Top Chord Bolt Shear OK"}, \text{"Top Chord Bolt Shear No Good"})$

$Check_{10} = \text{"Top Chord Bolt Shear OK"}$

- ***Check Slip Resistance Under Service II***

$P_{slip.top} := V_{bolt.serv} = 14.48\,kip$

$R_{n.slip.top} := K_h \cdot K_s \cdot N_s \cdot P_t = 19.5\,kip$ *Slip Resistance Per Bolt [AASHTO 6.13.2.8-1]*

$Check_{11} := if\left(P_{slip.top} < R_{n.slip.top}, \text{"Top Chord OK in Slip Resistance"}, \text{"Top Chord No Good in Slip Resistance"}\right)$

$Check_{11} = \text{"Top Chord OK in Slip Resistance"}$

- ***Check Fatigue at Bolts***

$\sigma_{top.fat} = 5.16 \cdot ksi$ *Fatigue Stress Range*

$F_{n.B} = 9.99 \cdot ksi$ *Fatigue Thershold [AASHTO Table 6.6.1.2.5-3, Category B]*

$Check_{12} := if\left(F_{n.B} > \sigma_{top.fat}, \text{"Top Chord Connection OK in Fatigue"}, \text{"Top Chord Connection No Good in Fatigue"}\right)$

$Check_{12} = \text{"Top Chord Connection OK in Fatigue"}$

Diagonal Chord Design Check:

- ***Member Forces***

$C_{diag} := \max\left(\left|C_{diag.mdx}\right|, P_{diag.str}, \left|1.4F_{diag}\right|\right) = 39.36\,kip$

Total Compression Force in Diagonal Member, max of Strength I or Strength 3

$T_{diag} := T_{diag.mdx} = 33.58\,kip$

Total Tension Force in Diagonal Member, Strength I

$F_{diag} := \max\left(\left|F_{diag.mdx}\right|, \left|P_{diag.ser2}\right|\right) = 29.4\,kip$

Max Force in Diagonal Member, Service II

$\sigma_{diag.fat} := \max\left(\sigma_{diag.fat.mdx}, \left|\frac{P_{diag.fat}}{A_{ang}}\right|\right) = 3\cdot ksi$

Fatigue Stress range in Member

- ***Member Capacity***
  - ***Compression***

$KLr_{eff} := \text{if}\left(\frac{L_{dg}}{r_{xy.ang}} \le 80, 72 + 0.75\cdot\frac{L_{dg}}{r_{xy.ang}}, 32 + 1.25\frac{L_{dg}}{r_{xy.ang}}\right) = 110.81$

Effective KL/r (AASHTO EQ 6.9.4.4-1 & 2), allows to only check flexural buckling failure mechanism - no LTB or combined force check needed

$Q := \begin{cases} 1 & \text{if } \frac{b_{f.ang}}{t_{f.ang}} \le 0.45\cdot\sqrt{\frac{E_s}{F_y}} \\ \text{"Slender"} & \text{otherwise} \end{cases} = 1$

Slender Element Reduction Factor [AASHTO 6.9.4.2.1-1]

$P_o := Q\cdot F_y\cdot A_i = 335\cdot kip$

Equivalent nominal yield resisitance [AASHTO 6.9.4.1.1]

$P_{e.ang} := \frac{\pi^2\cdot E_s}{\left(KLr_{eff}\right)^2}\cdot A_{ang} = 87.42\cdot kip$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

$P_{n.ang} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.ang}}}\cdot P_o\right) & \text{if } \frac{P_{e.ang}}{P_o} \ge 0.44 \\ \left(0.877\cdot P_{e.ang}\right) & \text{otherwise} \end{cases} = 76.67\cdot kip$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c\cdot P_{n.ang} = 72.83\,kip$

Factored Compressive Resistance of Angle

$Check_{13} := \text{if}\left(\phi_c\cdot P_{n.ang} \ge \left|C_{diag}\right|, \text{"Diagonal Chord OK in Compression"}, \text{"Diagonal Chord No Good in Compression"}\right)$

$Check_{13} = \text{"Diagonal Chord OK in Compression"}$

  - ***Tension***

$\phi P_{ny.ang} := \phi_y\cdot F_y\cdot A_{ang} = 178.12\,kip$

Factored Tensile Resistance in Yielding [6.8.2.1-1]

$U := 0.50$

Reduction Factor [6.8.2.2]

$\phi P_{nu} := \phi_u\cdot F_u\cdot A_{ang}\cdot U = 97.5\,kip$

Factored Tensile Resistance in Fracture [6.8.2.1-2]

$Check_{14} := \text{if}\left(\min\left(\phi P_{ny.ang}, \phi P_{nu}\right) \ge \left|T_{diag}\right|, \text{"Diagonal Chord OK in Tension"}, \text{"Diagonal Chord No Good in Tension"}\right)$

$Check_{14} = \text{"Diagonal Chord OK in Tension"}$

- ***Determine Weld Length Along Member***

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld (6.5.4.2)*

$\sigma_{diag.fat} = 3 \cdot ksi$ *Fatigue Stress Range*

$$L_{fat.req.w} := \frac{\sigma_{diag.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} = 5.1\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req} := \frac{\max(C_{diag}, T_{diag})}{\phi_{e2} \cdot 0.60F_{exx} \cdot 0.221in} = 5.3\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Weld_{diag} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$$

$$\boxed{Weld_{diag} = 10\,in}$$

- ***Determine Top Gusset Plate - to - Top Chord Weld***

Allowable Fatigue Stress based on equations 6.6.1.2.5-4:

$$L_{fat.req.t} := \frac{2\sigma_{diag.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} \qquad L_{fat.req.t} = 10.21\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req.t} := \frac{2\max(C_{diag}, T_{diag})}{\phi_u \cdot 0.60F_u \cdot 0.221in} \qquad L_{Str.req.t} = 11.42\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Req_Weld_Length_{tgus} := \max(L_{fat.req.t}, L_{Str.req.t})$$

$$\boxed{Req_Weld_Length_{tgus} = 11.42\,in}$$

- *Check Member Fatigue*

$$\sigma_{diag.fat.e} := \frac{\sigma_{diag.fat}}{1 - \dfrac{y_{o.ang}}{\max\left(b_{f.ang}, \dfrac{Weld_{diag} - b_{f.ang}}{2}\right)}} = 4.26 \cdot ksi$$

Effective fatigue stress on member (AASHTO Table 6.6.1.2.3-1, section 7.2)

$$Check := \text{if}\left(F_{n.E} > \sigma_{diag.fat.e}, \text{"Diagonal Chord Weld OK in Fatigue"}, \text{"Diaganol Chord Weld No Good in Fatigue"}\right)$$

$$\boxed{Check = \text{"Diagonal Chord Weld OK in Fatigue"}}$$

Bottom Chord Design Check:

- ***Member Forces***

$C_{bot} := \max\left(\left|C_{bot.mdx}\right|, \left|1.4 \cdot F_{bot.chord}\right|\right) = 13.09\,kip$

Total Compression Force in Bottom Member, max of Strength I or Strength III

$T_{bot} := T_{bot.mdx} = 2.39\,kip$

Total Tension Force in Bottom Member, Strength I

$F_{bot} := \max\left(\left|F_{bot.mdx}\right|, \left|P_{diag.ser2}\right| \cdot \cos(\theta)\right) = 23.68\,kip$

Max Force in Bottom Member, Service II

$\sigma_{bot.fat} := \max\left(\sigma_{bot.fat.mdx}, \left|\frac{P_{diag.fat}}{A_{ang}}\right| \cdot \cos(\theta)\right) = 2.42 \cdot ksi$

Fatigue Stress range in Member

- ***Member Capacity***
  - ***Compression***

$KLr_{eff} := \text{if}\left(\frac{L_h}{r_{xy.ang}} \le 80, 72 + 0.75 \cdot \frac{L_h}{r_{xy.ang}}, 32 + 1.25 \frac{L_h}{r_{xy.ang}}\right) = 144.38$

Effective KL/r (AASHTO EQ 6.9.4.4-1 & 2), allows to only check flexural buckling failure mechanism - no LTB or combined force check needed

$P_{e.ang} := \frac{\pi^2 \cdot E_s}{\left(KLr_{eff}\right)^2} \cdot A_{ang} = 51.49 \cdot kip$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

$P_{n.ang} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.ang}}} \cdot P_o\right) & \text{if } \frac{P_{e.ang}}{P_o} \ge 0.44 \\ \left(0.877 \cdot P_{e.ang}\right) & \text{otherwise} \end{cases} = 45.16 \cdot kip$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c \cdot P_{n.ang} = 42.9\,kip$

Factored Compressive Resistance of Angle

$Check_{15} := \text{if}\left(\phi_c \cdot P_{n.ang} \ge \left|C_{bot}\right|, \text{"Bottom Chord OK in Compression"}, \text{"Bottom Chord No Good in Compression"}\right)$

$Check_{15} = \text{"Bottom Chord OK in Compression"}$

  - ***Tension***

$\phi P_{ny.ang} := \phi_y \cdot F_y \cdot A_{ang} = 178.12\,kip$

Factored Tensile Resistance in Yielding [6.8.2.1-1]

$U := 0.50$

Reduction Factor [6.8.2.2]

$\phi P_{nu} := \phi_u \cdot F_u \cdot A_{ang} \cdot U = 97.5\,kip$

Factored Tensile Resistance in Fracture [6.8.2.1-2]

$Check_{16} := \text{if}\left(\min\left(\phi P_{ny.ang}, \phi P_{nu}\right) \ge \left|T_{bot}\right|, \text{"Bottom Chord OK in Tension"}, \text{"Bottom Chord No Good in Tension"}\right)$

$Check_{16} = \text{"Bottom Chord OK in Tension"}$

- *Determine Weld Length Along Member*

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld*

$\sigma_{bot.fat} = 2.42 \cdot ksi$ *Fatigue Stress Range*

$$L_{fat.req.w} := \frac{\sigma_{bot.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} = 4.11\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req} := \frac{\max(C_{bot}, T_{bot})}{\phi_u \cdot 0.60F_u \cdot 0.221in} = 1.9\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Weld_{bot} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$$

$$\boxed{Weld_{bot} = 10\,in}$$

- *Check Member Fatigue*

$$\sigma_{bot.fat.e} := \frac{\sigma_{bot.fat}}{1 - \frac{y_{o.ang}}{\max\left(b_{f.ang}, \frac{Weld_{bot} - b_{f.ang}}{2}\right)}} = 3.43 \cdot ksi$$

Effective fatigue stress on member (AASHTO Table 6.6.1.2.3-1, section 7.2)

$$Check := \text{if}(F_{n.E} > \sigma_{bot.fat.e}, \text{"Bottom Chord Weld OK in Fatigue"}, \text{"Bottom Chord Weld No Good in Fatigue"})$$

$$\boxed{Check = \text{"Bottom Chord Weld OK in Fatigue"}}$$

Check Bottom Gusset Plates:

Conservatively ignore skew for checking shear in bolts

$N_{bolts} := 5$

$I_p := \frac{N_{bolts}}{12} \cdot \left[S_{g.row}^2 \cdot \left(N_{bolts}^2 - 1 \right) \right]$ $\quad I_p = 90\,in^2$ *Bolt Pattern Inertia*

$ecc := 1.5 \cdot S_{g.row} = 4.5\,in$ *Eccentricity of Force on Bolt Group*

$y_{max} := \frac{S_{g.row}}{2} \cdot \left(N_{bolts} - 1 \right)$ $\quad y_{max} = 6\,in$ *Distance from Bolt Group CG to outer Bolt*

- ***Strength I: Compression in Diagonals, Tension in Bottom Chord***

$F_{x.bolts} := \left(C_{diag} \cdot \cos(\theta) - T_{bot} \right) \div N_{bolts} = 5.86\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := C_{diag} \cdot \sin(\theta) \div N_{bolts} = 4.66\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \frac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 8.79\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts} \right)^2 + F_{y.bolts}^2} = 15.38\,kip$ *Total Shear Force on Outer Bolt*

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Nominal Resistance Per Bolt*

$Check_{17} := if\left(V_{bolts} < \phi_s \cdot F_{v_bolt}, \text{"Bot Gusset Bolt Shear OK"}, \text{"Bot Gusset Bolt Shear No Good"} \right)$

$Check_{17} = \text{"Bot Gusset Bolt Shear OK"}$

- ***Bolt Bearing on Material***

$t_{mat} := \min\left(t_{pl}, t_{gus} \right)$ $\quad t_{mat} = 0.75\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min\left[\left(S_{g.row} - d_{h_stndrd} \right), \left(S_{pl.end} - 0.5 d_{h_stndrd} \right), \left(S_{g.end} - 0.5 d_{h_stndrd} \right) \right]$

$L_c = 1.03\,in$ *Minimum of Clear Distance Between Holes O Edge of Material in the Direction of Loading [6.13.2.9]*

$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2 d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases}$ $\quad R_n = 60.33\,kip$ *Nominal Resistance in Bearing, Per Bolt*

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 48.26\,kip$ *Resistance of Single Bolt in Bearing*

$Check_{18} := if\left(V_{bolts} < \phi_{bb} \cdot R_n, \text{"Bot Gusset Bolt Bearing OK"}, \text{"Bot Gusset Bolt Bearing No Good"} \right)$

$Check_{18} = \text{"Bot Gusset Bolt Bearing OK"}$

- ***Service II: Forces due to Axle Loads Included***

$F_{x.bolts} := \left(F_{diag} \cdot \cos(\theta) - F_{bot}\right) \div N_{bolts} = 0\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := F_{diag} \cdot \sin(\theta) \div N_{bolts} = 3.48\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \dfrac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 0\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\ 2}} = 3.48\,kip$

Total Shear Force on Outer Bolt

- ***Check Slip Resistance Under Service II***

$R_{n.slip.bot} := K_h \cdot K_s \cdot N_s \cdot P_t$ $R_{n.slip.bot} = 19.5\,kip$ *Nominal Slip REsistance per Bolt [AASHTO 6.13.2.8-1]*

$Check_{19} := \text{if}\left(V_{bolts} < R_{n.slip.bot}, \text{"Bot Gusset OK in Slip Resistance"}, \text{"Bot Gusset No Good in Slip Resistance"}\right)$

$Check_{19} = \text{"Bot Gusset OK in Slip Resistance"}$

- ***Fatigue: Forces due to Axle Loads Included***

$\sigma_{x.gus} := \left(\sigma_{diag.fat} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat} \cdot A_{ang}\right) \div \left(t_{gus} \cdot h_{gus.b}\right)$

$\sigma_{x.gus} = 0 \cdot ksi$ *Horizontal Force Acting on Group*

$\sigma_{m.gus} := \dfrac{\left(\sigma_{diag.fat} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat} \cdot A_{ang}\right) \cdot ecc \cdot y_{max}}{\dfrac{h_{gus.b}^{\ 3} \cdot t_{gus}}{12}}$ $\sigma_{m.gus} = 0 \cdot ksi$ *Stress in Extreme Fiber of Gusset Plate due to Eccentric Forces*

$\sigma_{gus.max} := \left|\sigma_{x.gus}\right| + \left|\sigma_{m.gus}\right|$ $\sigma_{gus.max} = 0 \cdot ksi$ *Max Stress in Plate*

$F_{th} := 16ksi$ *Allowable Fatigue Stress, Category B, [Table 6.6.1.2.5-3]*

$Check_{20} := \text{if}\left(F_{th} > \sigma_{gus.max}, \text{"Bot Gusset Connection OK in Fatigue"}, \text{"Bot Gusset Connection No Good in Fatigue"}\right)$

$Check_{20} = \text{"Bot Gusset Connection OK in Fatigue"}$

- ***Strength I: Tension in Diagonals, Compression in Bottom Chord***

$F_{x.bolts} := \left(T_{diag} \cdot \cos(\theta) - C_{bot}\right) \div N_{bolts} = 2.79\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := T_{diag} \cdot \sin(\theta) \div N_{bolts} = 3.98\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \dfrac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 4.19\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\ 2}} = 8.03\,kip$

Total Shear Force on Outer Bolt

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Nominal Resistance Per Bolt*

$Check_{21} := \text{if}\left(V_{bolts} < \phi_s \cdot F_{v_bolt}, \text{"Bot Gusset Bolt Shear OK"}, \text{"Bot Gusset Bolt Shear No Good"}\right)$

$Check_{21} = \text{"Bot Gusset Bolt Shear OK"}$

- ***Bolt Bearing on Material***

$t_{mat} := \min\left(t_{pl}, t_{gus}\right)$ $t_{mat} = 0.75\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min\left[\left(S_{g.row} - d_{h_stndrd}\right), \left(S_{pl.end} - 0.5d_{h_stndrd}\right), \left(S_{g.end} - 0.5d_{h_stndrd}\right)\right]$

$L_c = 1.03\,in$ *Minimum of Clear Distance Between Holes O Edge of Material in the Direction of Loading [6.13.2.9]*

$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases}$ $R_n = 60.33\,kip$ *Nominal Resistance in Bearing, Per Bolt*

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 48.26\,kip$ *Resistance of Single Bolt in Bearing*

$Check_{22} := \text{if}\left(V_{bolts} < \phi_{bb} \cdot R_n, \text{"Bot Gusset Bolt Bearing OK"}, \text{"Bot Gusset Bolt Bearing No Good"}\right)$

$Check_{22} = \text{"Bot Gusset Bolt Bearing OK"}$

- ***Service II: Without Forces Induced by Axle Loads***

$F_{x.bolts} := \left(F_{diag.mdx} \cdot \cos(\theta) - F_{bot.mdx}\right) \div N_{bolts}$ $\quad F_{x.bolts} = 2.61\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := F_{diag.mdx} \cdot \sin(\theta) \div N_{bolts}$ $\quad F_{y.bolts} = 2.95\,kip$ *Vertical Force Acting on Group, per bolt*

$$F_{mx.bolts} := \frac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p}$$ $\quad F_{mx.bolts} = 3.92\,kip$ *Force Due to Ecentricity on Outer Bolt*

$$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\;2}}$$

$V_{bolts} = 7.17\,kip$ *Total Shear Force on Outer Bolt*

- ***Check Slip Resistance Under Service II***

$R_{n.slip.bot} := K_h \cdot K_s \cdot N_s \cdot P_t$ $\quad R_{n.slip.bot} = 19.5\,kip$ *Nominal Slip REsistance per Bolt [AASHTO 6.13.2.8-1]*

$Check_{23} := \text{if}\left(V_{bolts} < R_{n.slip.bot}, \text{"Bot Gusset OK in Slip Resistance"}, \text{"Bot Gusset No Good in Slip Resistance"}\right)$

$Check_{23} = \text{"Bot Gusset OK in Slip Resistance"}$

- ***Fatigue: Without Forces Induced by Axle Loads***

$\sigma_{x.gus} := \left(\sigma_{diag.fat.mdx} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat.mdx} \cdot A_{ang}\right) \div \left(t_{gus} \cdot h_{gus.b}\right)$

$\sigma_{x.gus} = 0.15 \cdot ksi$ *Horizontal Force Acting on Group*

$$\sigma_{m.gus} := \frac{\left(\sigma_{diag.fat.mdx} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat.mdx} \cdot A_{ang}\right) \cdot ecc \cdot y_{max}}{\frac{h_{gus.b}^{\;3} \cdot t_{gus}}{12}}$$ $\quad \sigma_{m.gus} = 0.15 \cdot ksi$ *Stress in Extreme Fiber of Gusset Plate due to Eccentric Forces*

$\sigma_{gus.max} := \left|\sigma_{x.gus}\right| + \left|\sigma_{m.gus}\right|$ $\quad \sigma_{gus.max} = 0.29 \cdot ksi$ *Max Stress in Plate*

$F_{th} := 16ksi$ *Allowable Fatigue Stress, Category B, [Table 6.6.1.2.5-3]*

$Check_{24} := \text{if}\left(F_{th} > \sigma_{gus.max}, \text{"Bot Gusset Connection OK in Fatigue"}, \text{"Bot Gusset Connection No Good in Fatigue"}\right)$

$Check_{24} = \text{"Bot Gusset Connection OK in Fatigue"}$

Summary:

$$Summary_Checks := \begin{pmatrix} Check_1 \\ Check_2 \\ Check_3 \\ Check_4 \\ Check_5 \\ Check_6 \\ Check_7 \\ Check_8 \\ Check_9 \\ Check_{10} \\ Check_{11} \\ Check_{12} \\ Check_{13} \\ Check_{14} \\ Check_{15} \\ Check_{16} \\ Check_{17} \\ Check_{18} \\ Check_{19} \\ Check_{20} \\ Check_{21} \\ Check_{22} \\ Check_{23} \\ Check_{24} \end{pmatrix}$$

$$Summary_Checks = \begin{pmatrix} \text{"Cross Frame Depth OK"} \\ \text{"Bottom Chord Slenderness No Good"} \\ \text{"Top Chord Slenderness OK"} \\ \text{"Diagonal Chord Slenderness OK"} \\ \text{"Angle Plate Slenderness OK"} \\ \text{"Top Chord OK in Comp/Bend"} \\ \text{"Top Chord OK in Tension"} \\ \text{"Member OK in Fatigue"} \\ \text{"Top Chord Bolt Bearing OK"} \\ \text{"Top Chord Bolt Shear OK"} \\ \text{"Top Chord OK in Slip Resistance"} \\ \text{"Top Chord Connection OK in Fatigue"} \\ \text{"Diagonal Chord OK in Compression"} \\ \text{"Diagonal Chord OK in Tension"} \\ \text{"Bottom Chord OK in Compression"} \\ \text{"Bottom Chord OK in Tension"} \\ \text{"Bot Gusset Bolt Shear OK"} \\ \text{"Bot Gusset Bolt Bearing OK"} \\ \text{"Bot Gusset OK in Slip Resistance"} \\ \text{"Bot Gusset Connection OK in Fatigue"} \\ \text{"Bot Gusset Bolt Shear OK"} \\ \text{"Bot Gusset Bolt Bearing OK"} \\ \text{"Bot Gusset OK in Slip Resistance"} \\ \text{"Bot Gusset Connection OK in Fatigue"} \end{pmatrix}$$

Additional Top Gusset Bent Plate Checks:

$Check_{pry}$ = "No Bolt Prying will occur"

$Check_{pl}$ = "Bending in Gusset Plate Tip OK"

$Check_{pl.v}$ = "Vertical Bending in Plate OK"

Check = "Bottom Chord Weld OK in Fatigue"

Check = "Bottom Chord Weld OK in Fatigue"

Edge Bearing Cross Frame Design Checks

Ref: AASHTO LRFD, Sixth edition with 2014 Interims

- ***Methodology***
  1. *Use maximum of: axial forces output from MDX, forces induced by wheel loads over cross frame, and wind loads.*
  2. *Analyze/Design top chord for combination of axial force plus moment from edge beam load and wheel loads as if the member acts as a simply supported beam. Assume the concrete edge beam is dead load only.*

Geometry and Properties:

- ***Steel Properties***

$F_y := 50ksi$	*Yield Strength of Members (AASHTO M270, Gr 50)*
$F_u := 65ksi$	*Ultimate Strength of Members (AASHTO Table 6.4.1-1)*
$E_s := 29000ksi$	*Steel Modulus of Elasticity*
$F_{ub} := 120ksi$	*Ultimate Tensile Strength of Bolts (AASHTO 6.4.3.1) - AASHTO M164/ASTM A325, Type 3*
$F_{exx} := 70ksi$	*Strength of Weld Metal, Assumed*
$G := 11.2ksi$	*Shear modulus of Steel*

- ***Girder Properties***

$t_f := 1.25in$	*Bottom Flange Thickness*
$D_w := 60in$	*Web Depth*
$t_w := 0.75in$	*Web Thickness*
$S_{gird} := 7.5ft$	*Girder Spacing*
$t_{pl} := 1.0in$	*Connection Plate Thickness*
$b_{pl} := 9.5in$	*Connection Plate Width*
$skew := 45deg$	*Bearing to Girder Skew Angle*
$haunch := 4.5in$	*Clear distance between bottom of deck and bottom of top flange*

- ***Bolt Properties/Geometries***

$d_{bolt} := 0.875in$ *Diameter of H.S. Bolt for X-Frame Connection*

$d_{h_tens} := 0.9375in$ *Design Bolt Hole for Tension, Block Rupture [AASHTO 6.8.3 or 6.13.4]*

$d_{h_stndrd} := d_{bolt} + 0.0625in$ $\quad d_{h_stndrd} = \frac{15}{16}in$ *Standard Bolt Hole Diameter [6.13.2.4]*

$S_{g.end} := 1.50in$ $\quad S_{pl.end} := 1.5in$ $\quad S_{g.row} := 3.0in$ *Bolt Hole Geometry, See Diagram*

$S_{t2} := 3.0in$ $\quad S_{t.end} := 1.50in$ $\quad S_{t.pl} := 1.5in$ $\quad S_{t1} := 3.0in$ *Top connection dimensions, see diagram*

$A_b := \pi \cdot (0.5 \cdot d_{bolt})^2$ $\quad A_b = 0.601 \cdot in^2$ *Area of Single Bolt*

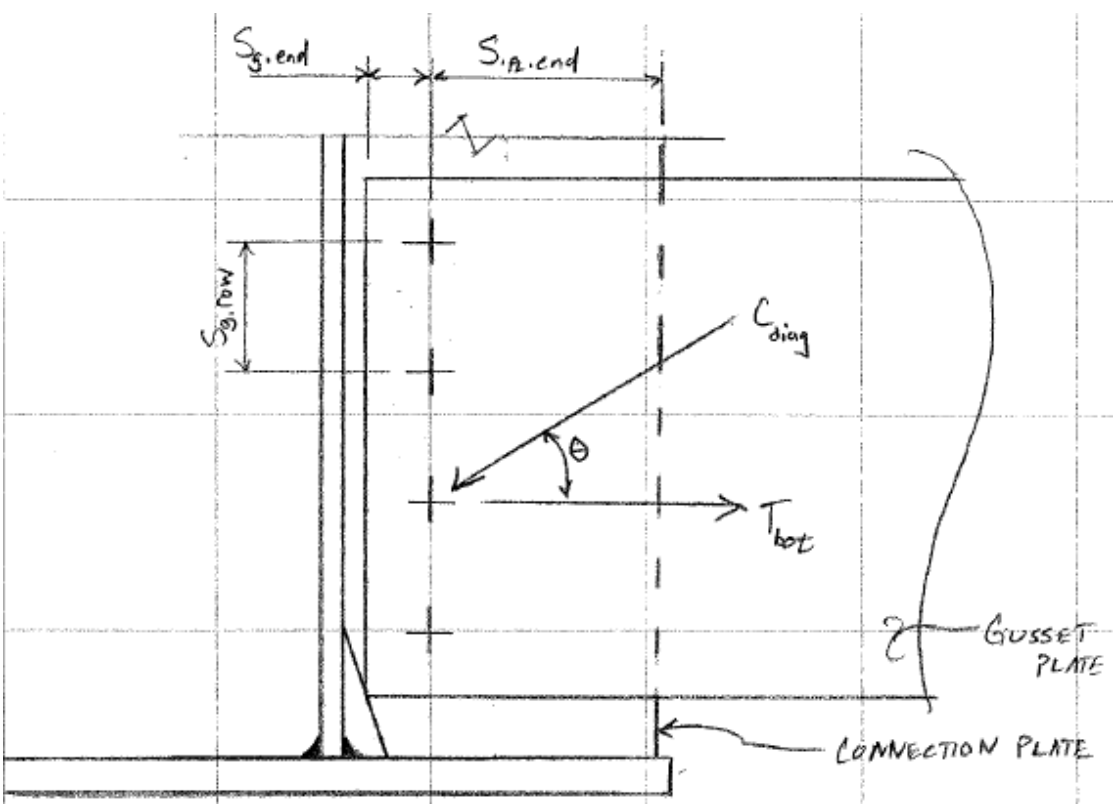
$P_t := 39kip$ *Minimum Bolt Tension (AASHTO Table 6.13.2.8-1)*

$K_h := 1.0$ *Hole Size Factor (AASHTO Table 6.13.2.8-2)*

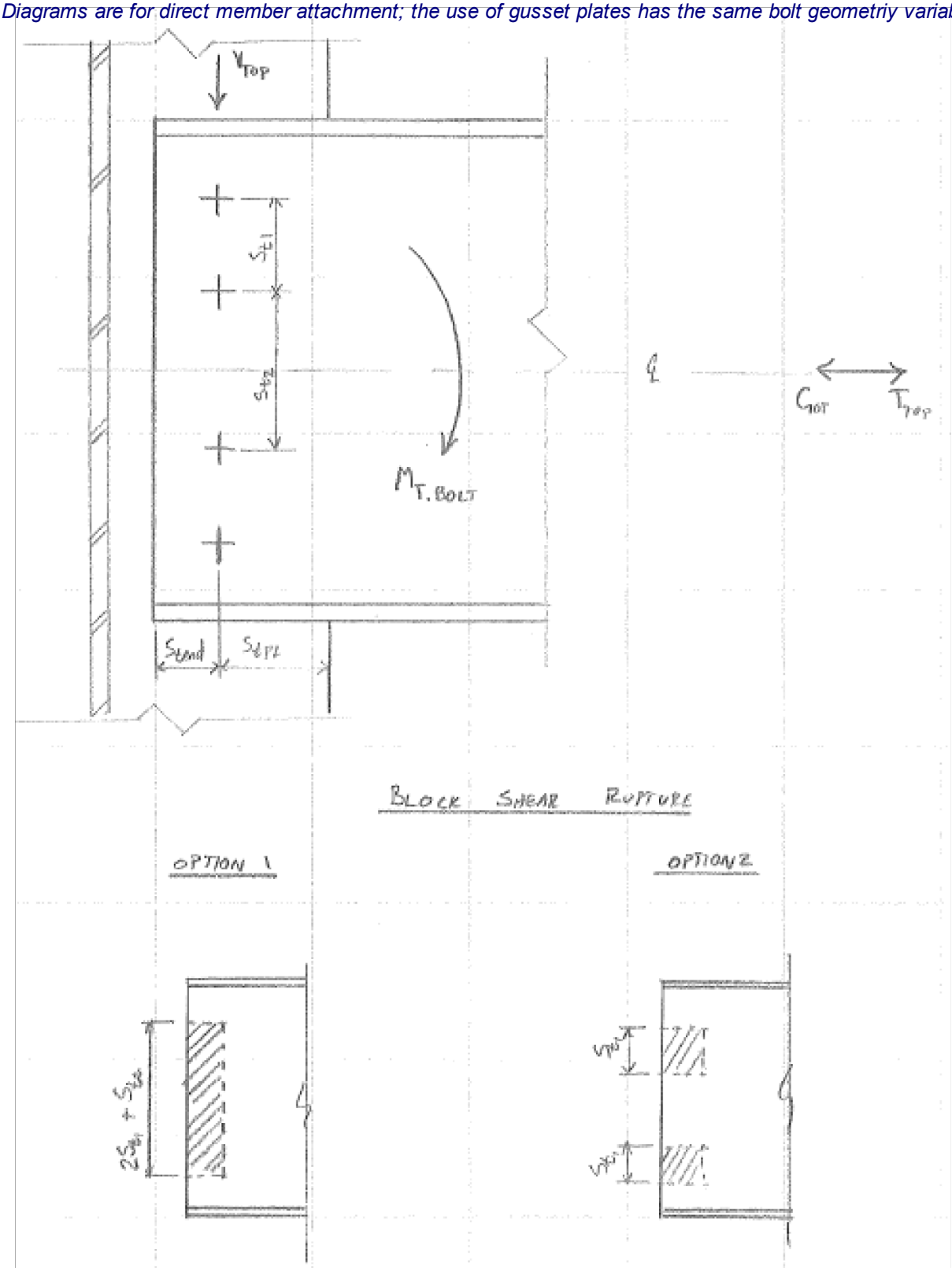
$K_s := 0.50$ *Surface Condition Factor (AASHTO Table 6.13.2.8-3) - Class B Surface*

$N_s := 1$ *Number of Slip Planes*

$N_{bolts} := 4$ *Number of Bolts per Connection*



Diagrams are for direct member attachment; the use of gusset plates has the same bolt geometriy variables.



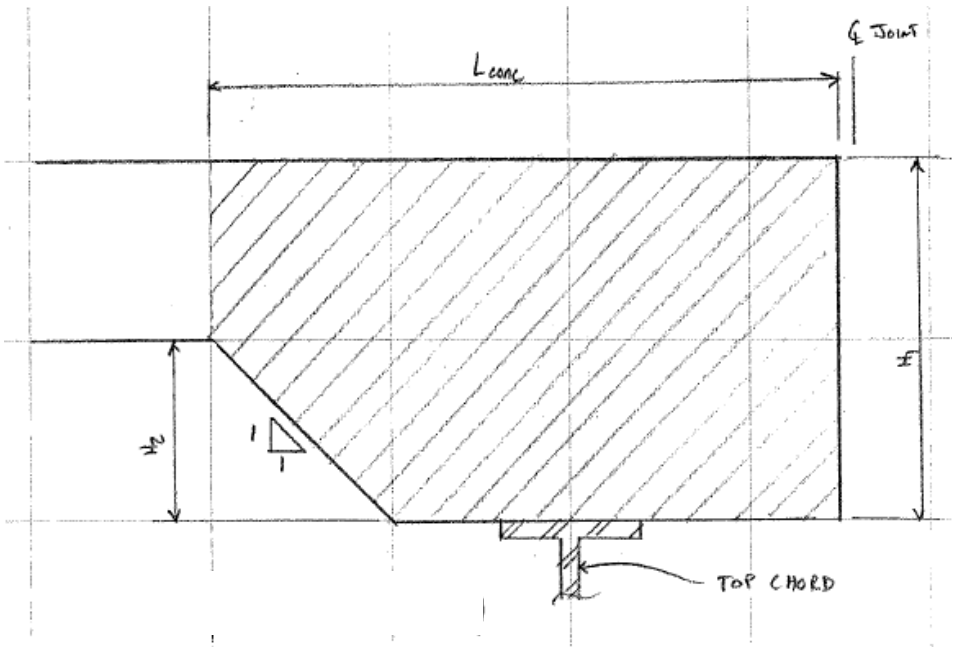
- ***Concrete Edge Beam Geometry***

$H_1 := 9.0in + haunch + 4in = 17.50\,in$ — *Total Depth of Beam (deck+haunch+offset below flange)*

$H_2 := H_1 - 9.0in = 8.5\,in$ — *Depth of Beam Beyond Deck Thickness*

$L_{conc} := 2 \cdot 13in + 0in = 26\,in$ — *Width of Beam Supported by Top Chord (No 1:1 Bevel included)*

$A_{conc} := H_1 \cdot L_{conc}$ $\quad A_{conc} = 3.16 \cdot ft^2$ — *Area of Beam*



- ***Cross Frame Geometry***

Approximate for calculation purposes only

$H_{xf} := Round(D_w - 7.5in - 5.5in, 1in)$ $\quad H_{xf} = 47\,in$ — *Height Between CL Top & Bot Chord*

$W_{xf} := \frac{S_{gird}}{\cos(skew)} - \frac{t_w}{\cos(skew)} - \frac{(b_{pl})}{\cos(skew)} - 4in$ $\quad W_{xf} = 108.78\,in$ — *Horizontal Dist Between Center of connection of top/bottom chord*

$\theta := atan\left[\frac{H_{xf}}{0.5W_{xf} + (b_{pl})}\right]$ $\quad \theta = 36.34 \cdot deg$ — *Angle Between Members*

$H_{dg} := 31in$ $\quad H_{dg} = 31\,in$ — *Vertical Height of Diagonals*

$L_h := W_{xf}$ $\quad L_h = 108.78\,in$ — *Length of Top and Bot Chords*

$L_{dg} := \sqrt{H_{dg}^2 + (0.5W_{xf})^2}$ $\quad L_{dg} = 62.61\,in$ — *Length of Diagonals*

$t_{gus} := 0.75in$ — *Thickness of Gusset Plates*

$h_{gus.b} := 18in$ — *Height of Bottom Gusset Plates*

$h_{gus.t} := 12in$ — *Height of Top Gusset Plates*

- *L4x4x1/2 Properties (AISC Table 1-7)*

$A_{ang} := 3.75in^2$

$b_{f.ang} := 4in$

$t_{f.ang} := 0.5in$

$r_{xy.ang} := 1.21in$

$r_{z.ang} := 0.776in$

$J_{ang} := 0.322in^4$

$I_{y.ang} := 5.52in^4$

$S_{x.ang} := 1.96in^3$

$y_{o.ang} := 1.18in$

- *MC8x22.8 Properties (AISC Table 1-6)*

$A_i := 6.7in^2$

$d_i := 8.0in$

$t_{w.i} := 0.427in$

$k_i := 1.1875in$

$b_{f.i} := 3.50in$

$t_{f.i} := 0.525in$

$r_{y.i} := 1.02in$

$r_{x.i} := 3.09in$

$y_o := 1.04in$

$I_{x.i} := 63.8in^4$

$I_{y.i} := 7.01in^4$

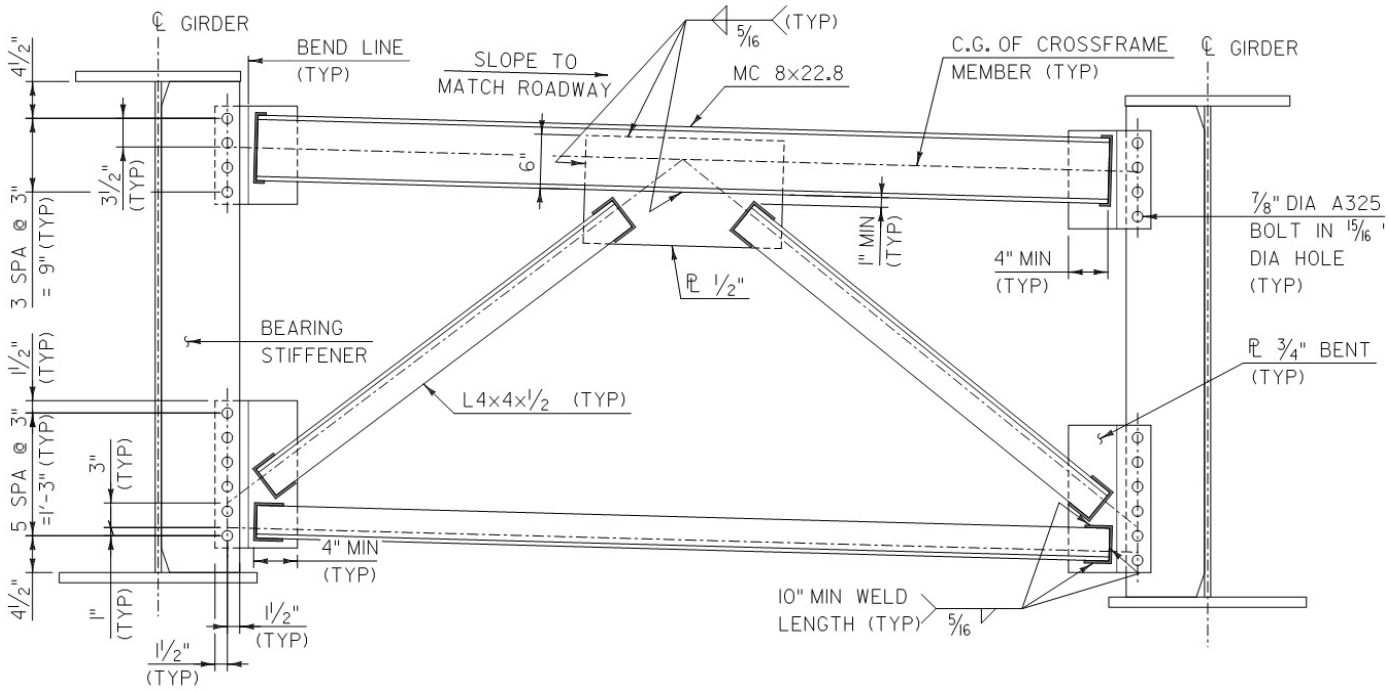
$Z_{x.i} := 19.1in^3$

$Z_{y.i} := 5.37in^3$

$S_{x.i} := 15.8in^3$

$J_i := 0.572in^4$

$C_{w.i} := 75.2in^6$



ABUTMENT 2 END CROSSFRAME DETAIL

SCALE: 1 1/2" = 1'-0"

Wind Forces:

Computation of Wind Forces in CF members:

Wind pressure $p_w := 50.0psf$ *AASHTO LRFD Table 3.8.1.2.1-1*

Tributary length for abutment $L_{trib} := \frac{172ft}{2} = 86 \cdot ft$

Superstructure height: $H := 7in + 9in + haunch + 60in + 1.75in = 6.85 \cdot ft$

$H_{misc} := (8 + 4)in = 12\,in$ *from horizontal rails (approx.)*

$H_{tot} := H + H_{misc} = 7.85 \cdot ft$ *exposed height for wind*

Wind load on bottom flange at abutment CF

$F_w := \frac{-p_w \cdot L_{trib} \cdot H_{tot}}{2} = -16.89 \cdot kip$ *Compression load, the other half goes thru deck slab*

$F_{diag} := F_w \cdot \left(\frac{\cos(\theta)}{1 + \cos(\theta)}\right) \cdot \frac{1}{\cos(\theta)} = -9.35\,kip$ *Axial componenet of portion of wind load carried by diagonal*

$F_{bot.chord} := F_w \cdot \frac{1}{1 + \cos(\theta)} = -9.35\,kip$ *Portion of wind load carried by bottom chord*

$F_{top.chord} := 0kip$ *No additional loads in top chord due to balancing of diagonals.*

MDX Member Forces:

- *Maximum Factored Loads Among All End Bearing Cross Frames, Taken from MDX Output and/or Excel*

$T_{top} := 26.08kip$ *Tension in Top Chord*

$C_{top} := -23.6kip$ *Compression in Top Chord*

$T_{bot.mdx} := 2.39kip$ *Tension in Bottom Chord*

$C_{bot.mdx} := -9.44kip$ *Compression in Bottom Chord*

$T_{diag.mdx} := 33.58kip$ *Tension in Diagonal Chord*

$C_{diag.mdx} := -33.58kip$ *Compression in Diagonal Chord*

- *Fatigue Stress Range*

$\sigma_{top.fat.mdx} := 1.77ksi$ *Fatigue Stress Range in Top Chord*

$\sigma_{bot.fat.mdx} := 1.58ksi$ *Fatigue Stress Range in Bottom Chord*

$\sigma_{diag.fat.mdx} := 2.61ksi$ *Fatigue Stress Range in Diagonal Chord*

- *Service II Forces*

$F_{top.mdx} := 19.34kip$ *Absolute Max Force in Top Chord*

$F_{bot.mdx} := 7.01kip$ *Absolute Max Force in Bottom Chord*

$F_{diag.mdx} := 24.92kip$ *Absolute Max Force in Diagonal Chord*

Fatigue Thresholds:

Considering low traffic count, use Fatigue II load combination (finite fatigue life)

$n := 1.0$ *Cycles per truck passage*

$ADTT := 330$ *Single lane traffic count, extrapolated to 50 year mark*

$N := 365 \cdot 100 \cdot n \cdot ADTT = 1.2 \times 10^7$ *Total cycles, assuming 100 year life*

$A_B := 120 \cdot 10^8 (ksi)^3$ *Fatigue Detail Category Constant, Category B (AASHTO Table 6.6.1.2.5-1)*

$A_E := 11 \cdot 10^8 (ksi)^3$ *Fatigue Detail Category Constant, Category E (AASHTO Table 6.6.1.2.5-1)*

$$F_{n.B} := \left(\frac{A_B}{N}\right)^{\left(\frac{1}{3}\right)} = 9.99 \cdot ksi$$

Allowable Fatigue Stress Rabge for Category B Details (AASHTO 6.6.1.2.5-2)

$$F_{n.E} := \left(\frac{A_E}{N}\right)^{\left(\frac{1}{3}\right)} = 4.5 \cdot ksi$$

Allowable Fatigue Stress Rabge for Category E Details (AASHTO 6.6.1.2.5-2)

Geometry Checks:

$$Check_1 := if\left[H_{xf} + \frac{d_i}{2} + \frac{b_{f.ang}}{2} \geq 0.75\left(D_w + 2t_f\right), \text{"Cross Frame Depth OK"}, \text{"Cross Frame Depth Needs to Be Increased"}\right]$$

$Check_1 = \text{"Cross Frame Depth OK"}$ *[AASHTO 6.7.4.2]*

- *Slenderness Checks*

$K := 0.75$

$K_{ang} := 1.0$

Effective Length Factors (4.6.2.5)

$KL_R_{all} := 140$ *Allowable Slenderness [6.9.3]*

$K_{ang} \cdot L_h \div \min\left(r_{xy.ang}, r_{z.ang}\right) = 140.18$ *Bottom Chord Slenderness Ratio [6.9.3]*

$K_{ang} \cdot L_{dg} \div \min\left(r_{xy.ang}, r_{z.ang}\right) = 80.68$ *Diag Member Slenderness Ratio [6.9.3]*

$K \cdot \frac{L_h}{2} \div \min\left(r_{x.i}, r_{y.i}\right) = 39.99$ *Top Chord Slenderness Ratio [6.9.3]*

$$Check_2 := if\left(\frac{K_{ang} \cdot L_h}{\min\left(r_{xy.ang}, r_{z.ang}\right)} < KL_R_{all}, \text{"Bottom Chord Slenderness OK"}, \text{"Bottom Chord Slenderness No Good"}\right)$$

$Check_2 = \text{"Bottom Chord Slenderness No Good"}$ *Say OK - consistent with original design and only over the limit by 0.1%*

$$Check_3 := if\left(\frac{K \cdot L_h}{2\min\left(r_{x.i}, r_{y.i}\right)} < KL_R_{all}, \text{"Top Chord Slenderness OK"}, \text{"Top Chord Slenderness No Good"}\right)$$

$Check_3 = \text{"Top Chord Slenderness OK"}$

$$Check_4 := if\left(\frac{K_{ang} \cdot L_{dg}}{\min\left(r_{xy.ang}, r_{z.ang}\right)} < KL_R_{all}, \text{"Diagonal Chord Slenderness OK"}, \text{"Diag Chord Slenderness No Good"}\right)$$

$Check_4 = \text{"Diagonal Chord Slenderness OK"}$

$$Check_5 := if\left(\frac{b_{f.ang}}{t_{f.ang}} < 0.45\sqrt{\frac{E_s}{F_y}}, \text{"Angle Plate Slenderness OK"}, \text{"Angle Leg is Too Slender, No Good"}\right)$$

$Check_5 = \text{"Angle Plate Slenderness OK"}$ *[AASHTO 6.9.4.2-1]*

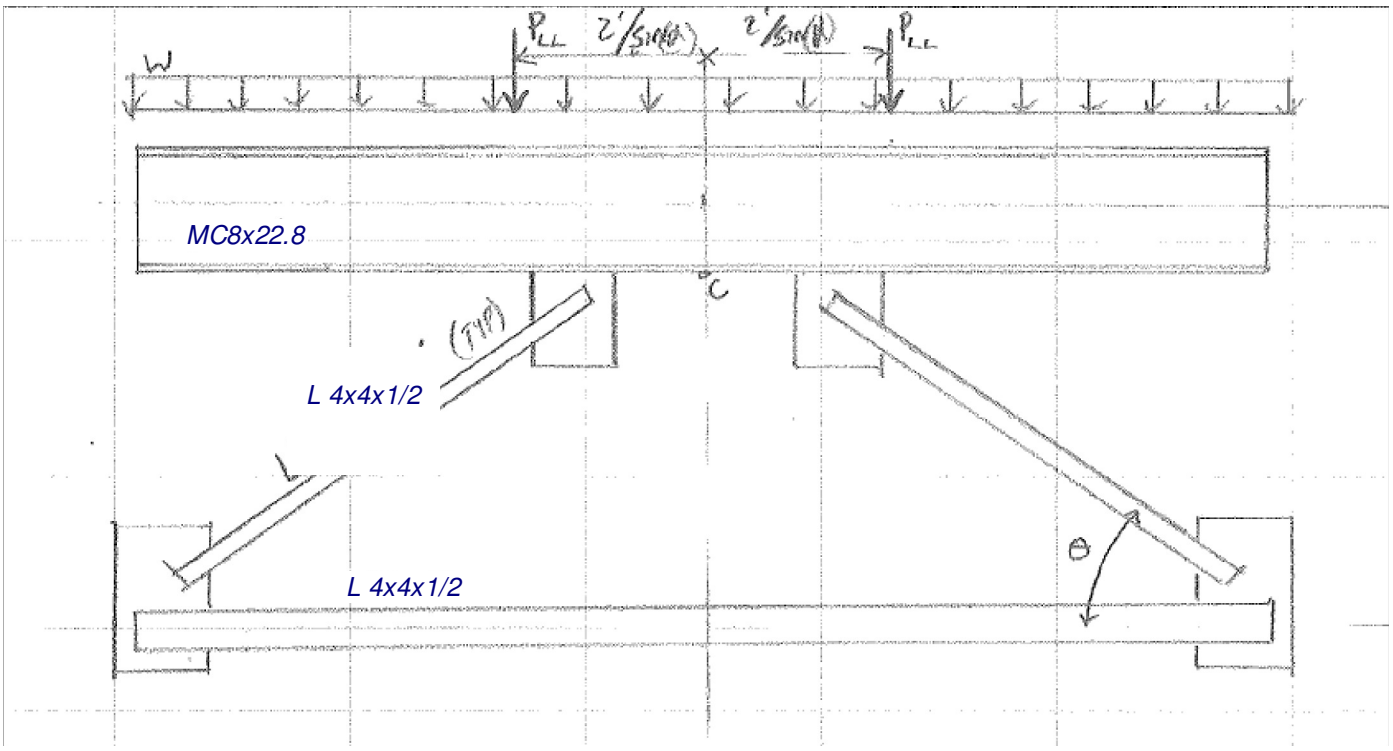
Bending Loads:

For force distribution, use deflection compatability and superposition to find equivalent center deflection due to gravity loads and equivalent stiffness of diagonal members pushing vertically.

$w_{conc} := 150pcf \cdot A_{conc}$ $\quad w_{conc} = 473.96 \cdot plf$ *Concrete Edge Beam Dist Load*

$w_{self} := A_i \cdot 490pcf$ $\quad w_{self} = 22.8 \cdot plf$ *Self weight of Top Chord*

$P_{LL} := 16kip$ *Live Load Wheel Load*



- ***Strength I***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.2 \cdot 1.33 \cdot 1.75 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} + \frac{5\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$a := \mathrm{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right)$ $\delta_{str1} := a_0 \cdot in = 0.023 \cdot in$ $P_{str.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 39.36 kip$ *Midspan deflection and axial force in diagonals*

$M_{Str1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{str1}}{L_h^2} = 3.54 \cdot kip \cdot ft$ *Moment in Top Chord*

$V_{Str.1} := 0.5\left[1.2 \cdot 1.33 \cdot 1.75 P_{LL} + \left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 1.84 kip$ *Shear at ends of Chord*

- *Wheel loads from two passing trucks*

$b := \left(\frac{L_h}{2} - \frac{2ft}{\cos(skew)}\right)$ *Distance from beam support to wheel load application*

guess $\delta_2 := 0.03in$ $P_{s2} := 6kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.33 \cdot 1.75 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] + \frac{5\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$a := \mathrm{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right)$ $\delta_{str2} := a_0 \cdot in = 0.02 \cdot in$ $P_{str.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 35.55 kip$ *Midspan deflection and axial force in diagonals*

$M_{Str2.pos} := 1.33 \cdot 1.75 P_{LL} \cdot (b) + \frac{\left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot (b)}{2} \cdot \left[L_h - (b)\right] - \frac{a_1 \cdot kip \cdot (b)}{2} = 31.46 \cdot kip \cdot ft$ *Max Pos Moment*

$M_{Str2.neg} := 1.33 \cdot 1.75 P_{LL} \cdot (b) + \frac{\left(1.25 w_{conc} + 1.25 w_{self}\right)}{8} \cdot \left(L_h^2\right) - \frac{a_1 \cdot kip \cdot \left(L_h\right)}{4} = -25.64 \cdot kip \cdot ft$ *Max Neg Moment*

$V_{Str.2} := 0.5\left[2 \cdot 1.33 \cdot 1.75 P_{LL} + \left(1.25 w_{conc} + 1.25 w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 18.99 kip$ *Shear at ends of Chord*

- *Controlling Forces*

$M_{u.Str1} := \max\left(M_{Str1}, M_{Str2.pos}, \left|M_{Str2.neg}\right|\right) = 31.46 \cdot kip \cdot ft$ *Maximum Moment in Top Chord*

$P_{diag.str} := \max\left(P_{str.1}, P_{str.2}\right) = 39.36 kip$ *Maximum Axial load in diagonals*

$V_{u.Str} := \max\left(V_{Str.1}, V_{Str.2}\right) = 18.99 kip$ *Maximum Shear at ends of Top Chord*

- ***Service II***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.2 \cdot 1.33 \cdot 1.30 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} + \frac{5\left(w_{conc} + w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$

Midspan deflection in Top Chord

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$

Deflection in diagonals (treated as a spring)

$$a := \text{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right) \quad \delta_{ser1} := a_0 \cdot in = 0.017 \cdot in \quad P_{ser.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 29.4\,kip$$

Midspan deflection and axial force in diagonals

$$M_{Ser1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{ser1}}{L_h^2} = 2.65 \cdot kip \cdot ft$$

Moment in Top Chord

$$V_{Ser.1} := 0.5\left[1.2 \cdot 1.33 \cdot 1.30 P_{LL} + \left(w_{conc} + w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 1.43\,kip$$

Shear at ends of Chord

- *Wheel loads from two passing trucks*

$$b := \left(\frac{L_h}{2} - \frac{2ft}{\cos(skew)}\right)$$

Distance from beam support to wheel load application

guess $\delta_2 := 0.03in$ $P_{s2} := 6kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.33 \cdot 1.30 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] + \frac{5\left(w_{conc} + w_{self}\right) \cdot \left(L_h^4\right)}{384 \cdot E_s \cdot I_{x.i}} - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$

Midspan deflection in Top Chord

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$

Deflection in diagonals (treated as a spring)

$$a := \text{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right) \quad \delta_{ser2} := a_0 \cdot in = 0.015 \cdot in \quad P_{ser.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 26.57\,kip$$

Midspan deflection and axial force in diagonals

$$M_{Ser2.pos} := 1.33 \cdot 1.30 P_{LL} \cdot (b) + \frac{\left(w_{conc} + w_{self}\right) \cdot (b)}{2} \cdot \left[L_h - (b)\right] - \frac{a_1 \cdot kip \cdot (b)}{2} = 23.43 \cdot kip \cdot ft$$

Max Pos Moment

$$M_{Ser2.neg} := 1.33 \cdot 1.30 P_{LL} \cdot (b) + \frac{\left(w_{conc} + w_{self}\right)}{8} \cdot \left(L_h^2\right) - \frac{a_1 \cdot kip \cdot \left(L_h\right)}{4} = -19.12 \cdot kip \cdot ft$$

Max Neg Moment

$$V_{Ser.2} := 0.5\left[2 \cdot 1.33 \cdot 1.30 P_{LL} + \left(w_{conc} + w_{self}\right) \cdot \left(L_h\right) - a_1 \cdot kip\right] = 14.17\,kip$$

Shear at ends of Chord

- *Controlling Forces*

$$M_{Serv2} := \max\left(M_{Ser1}, M_{Ser2.pos}, \left|M_{Ser2.neg}\right|\right) = 23.43 \cdot kip \cdot ft$$

Maximum Moment in Top Chord

$$P_{diag.ser2} := \max\left(P_{ser.1}, P_{ser.2}\right) = 29.4\,kip$$

Maximum Axial load in diagonals

$$V_{u.Ser2} := \max\left(V_{Ser.1}, V_{Ser.2}\right) = 14.17\,kip$$

Maximum Shear at ends of Top Chord

- ***Fatigue***
- *Single Wheel Load on Center*

guess $\delta_1 := 0.02in$ $P_{s1} := 30kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_1 = \frac{1.15 \cdot 0.75 P_{LL} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}} - \frac{P_{s1} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_1 = \frac{P_{s1} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$$a := \text{Find}\left(\frac{\delta_1}{in}, \frac{P_{s1}}{kip}\right) \quad \delta_{fat1} := a_0 \cdot in = 0.006 \cdot in \quad P_{fat.1} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 11.27\,kip$$ *Midspan deflection and axial force in diagonals*

$$M_{fat1} := \frac{12 \cdot E_s \cdot I_{x.i} \cdot \delta_{fat1}}{L_h^2} = 1.01 \cdot kip \cdot ft$$ *Moment in Top Chord*

$$V_{fat.1} := 0.5\left(1.15 \cdot 1.50 P_{LL} - a_1 \cdot kip\right) = 7.12\,kip$$ *Shear at ends of Chord*

- *Wheel loads from single passing trucks, centered over bay*

$$b := \left(\frac{L_h}{2} - \frac{3ft}{\cos(skew)}\right)$$ *Distance from beam support to wheel load application*

guess $\delta_2 := 0.003in$ $P_{s2} := 5kip$ *Guess at total deflection and equivalent "spring force" from angles*

Given

$$\delta_2 = \frac{1.15 \cdot 0.75 P_{LL} \cdot (b)}{24 \cdot E_s \cdot I_{x.i}} \cdot \left[3 L_h^2 - 4 (b)^2\right] - \frac{P_{s2} \cdot L_h^3}{48 \cdot E_s \cdot I_{x.i}}$$ *Midspan deflection in Top Chord*

$$\delta_2 = \frac{P_{s2} \cdot L_{dg}}{2 A_{ang} \cdot E_s \cdot \sin(\theta)}$$ *Deflection in diagonals (treated as a spring)*

$$a := \text{Find}\left(\frac{\delta_2}{in}, \frac{P_{s2}}{kip}\right) \quad \delta_{fat2} := a_0 \cdot in = 0.001 \cdot in \quad P_{fat.2} := \frac{a_1 \cdot kip}{2 \cdot \sin(\theta)} = 2.16\,kip$$ *Midspan deflection and axial force in diagonals*

$$M_{fat2.pos} := 1.15 \cdot 0.75 P_{LL} \cdot (b) - \frac{a_1 \cdot kip \cdot (b)}{2} = 3.63 \cdot kip \cdot ft$$ *Max Pos Moment*

$$M_{fat2.neg} := 1.15 \cdot 0.75 P_{LL} \cdot (b) - \frac{a_1 \cdot kip \cdot (L_h)}{4} = -1.8 \cdot kip \cdot ft$$ *Max Neg Moment*

$$V_{fat.2} := 0.5\left(2 \cdot 1.15 \cdot 0.75 P_{LL} - a_1 \cdot kip\right) = 12.52\,kip$$ *Shear at ends of Chord*

- *Controlling Forces*

$$M_{Fat} := \max\left(M_{fat1}, M_{fat2.pos}, \left|M_{fat2.neg}\right|\right) = 3.63 \cdot kip \cdot ft$$ *Maximum Moment in Top Chord*

$$P_{diag.fat} := \max\left(P_{fat.1}, P_{fat.2}\right) = 11.27\,kip$$ *Maximum Axial load in diagonals*

$$V_{u.fat} := \max\left(V_{fat.1}, V_{fat.2}\right) = 12.52\,kip$$ *Maximum Shear at ends of Top Chord*

Top Chord Design Checks:

- ***Compression/Bending***

  - *Nominal Design Load*

$\phi_c := 0.95$ *Resistance Factor [6.5.4.2]*

$$Q := \begin{cases} 1 & \text{if } \frac{b_{f.i}}{t_{f.i}} \le 0.56\cdot\sqrt{\frac{E_s}{F_y}} \wedge \frac{d_i - 2\cdot k_i}{t_{w.i}} \le 1.49\cdot\sqrt{\frac{E_s}{F_y}} \\ \text{"Slender"} & \text{otherwise} \end{cases} = 1$$

Slender Element Reduction Factor [AASHTO 6.9.4.2.1-1]

$P_o := Q\cdot F_y\cdot A_i = 335\cdot kip$ *Equivalent nominal yield resisitance [AASHTO 6.9.4.1.1]*

Flexural Buckling

$$P_{e.FB} := \frac{\pi^2\cdot E_s}{\left(\frac{K\cdot L_h \div 2}{\min(r_{x.i}, r_{y.i})}\right)^2}\cdot A_i = 1198.9\cdot kip$$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

Flexural Torsional Buckling

$$P_{ex} := \frac{\pi^2\cdot E_s}{\left(\frac{K\cdot L_h \div 2}{r_{x.i}}\right)^2}\cdot A_i = 1.1\times 10^4\,kip$$

AASHTO 6.9.4.1.3-4

$$r_o := \sqrt{y_o^2 + \frac{I_{x.i} + I_{y.i}}{A_i}} = 3.41\,in$$

Polar Radius of Inertia (AASHTO 6.9.4.1.3-6)

$$P_{ez} := \left[\frac{\pi^2\cdot E_s\cdot C_{w.i}}{(K\cdot L_h \div 2)^2} + G\cdot J_i\right]\cdot\frac{1}{r_o^2} = 1110.72\,kip$$

AASHTO 6.9.4.1.3-5

$$H_i := 1 - \frac{y_o^2}{r_o^2} = 0.91$$

AASHTO 6.9.4.1.3-3

$$P_{e.FTB} := \frac{P_{ex} + P_{ez}}{2H_i}\cdot\left[1 - \sqrt{1 - \frac{4P_{ex}\cdot P_{ez}\cdot H_i}{(P_{ex} + P_{ez})^2}}\right] = 1099.39\,kip$$

Elastic Buckling base on FTB, (AASHTO 6.9.4.1.3-2)

$$P_{e.i} := \min(P_{e.FB}, P_{e.FTB}) = 1099.39\,kip$$

$$P_{n.i} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.i}}}\cdot P_o\right) & \text{if } \frac{P_{e.i}}{P_o} \ge 0.44 \\ (0.877\cdot P_{e.i}) & \text{otherwise} \end{cases} = 294.89\cdot kip$$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c \cdot P_{n.i} = 280.14\,kip$ *Factored Compressive Resistance*

$M_{uy_Str} := \left|C_{top}\right| \cdot (y_o) = 2.05 \cdot kip \cdot ft$ *Bending about the y-axis due to eccentric compression load*

$M_{ry} := F_y \cdot Z_{y.i} = 22.37 \cdot kip \cdot ft$ *Moment capacity about the y-axis, fully braced on top, mid point brace on bottom, therefore assume full capacity can be reached.*

$M_r := F_y \cdot Z_{x.i}$ $M_r = 79.58 \cdot kip \cdot ft$ *Resistive moment, noncomposite, fully braced compresion flange.*

$$Bend_Ratio := \begin{cases} \dfrac{\left|C_{top}\right|}{2(\phi_c \cdot P_{n.i})} + \dfrac{M_{u.Str1}}{M_r} + \dfrac{M_{uy_Str}}{M_{ry}} & \text{if } \dfrac{\left|C_{top}\right|}{\phi_c \cdot P_{n.i}} < 0.2 \\ \dfrac{\left|C_{top}\right|}{\phi_c \cdot P_{n.i}} + \dfrac{8}{9}\left(\dfrac{M_{u.Str1}}{M_r} + \dfrac{M_{uy_Str}}{M_{ry}}\right) & \text{otherwise} \end{cases}$$

[6.9.2.2-1]

[6.9.2.2-2]

$Bend_Ratio = 0.53$

$Check_6 := \text{if}(Bend_Ratio < 1.0, \text{"Top Chord OK in Comp/Bend"}, \text{"Top Chord No Good in Comp/Bend"})$

$Check_6 = \text{"Top Chord OK in Comp/Bend"}$

- ***Tension (w/o Bending)***
  - *Yielding and Fracture*

$\phi_y := 0.95$ *Resistance Factor for Yielding [6.5.4.2]*

$\phi_u := 0.80$ *Resistance Factor for Fracture [6.5.4.2]*

$U := 0.75$ *Reduction Factor [6.8.2.2]*

$A_{n.i} := A_i = 6.7\,in^2$ *Net Area of Member in Tension*

$\phi P_{ny} := \phi_y \cdot F_y \cdot A_i = 318.25\,kip$ *Factored Tensile Resistance in Yielding [6.8.2.1-1]*

$\phi P_{nu} := \phi_u \cdot F_u \cdot A_{n.i} \cdot U = 261.3\,kip$ *Factored Tensile Resistance in Fracture [6.8.2.1-2]*

$\phi P_{r.tens} := \min(\phi P_{ny}, \phi P_{nu}) = 261.3\,kip$ *Factored Tensile Resistance [6.8.2.1]*

$Check_7 := if(\phi P_{r.tens} \geq |T_{top}|, \text{"Top Chord OK in Tension"}, \text{"Top Chord No Good in Tension"})$

$Check_7 = \text{"Top Chord OK in Tension"}$

- ***Determine Weld Length Along Member***

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld*

$F_{n.B} = 9.99 \cdot ksi$ *Fatigue Threshold in Weld Matal [AASHTO 6.6.1.2.5-3, Category B]*

$L_{fat.req.w} := \dfrac{\sigma_{top.fat.mdx} \cdot A_i}{F_{n.B} \cdot 0.221in} = 5.37\,in$ *Required Length of Weld, Based on Fatigue inthe weld metal and assuming a 5/16" weld size*

$L_{Str.req} := \dfrac{\max(|C_{top}|, T_{top}, V_{u.Str})}{\phi_{e2} \cdot 0.60 F_{exx} \cdot 0.221in} = 3.51\,in$ *Required Length of Weld, Based on Strength I and assuming a 5/16" weld size*

$Weld_{top} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$ $Weld_{top} = 10\,in$ *Ten inches included to match VTrans plans*

- ***Check Fatigue in Member***

Away from welds

$\sigma_{top.fat} := \sigma_{top.fat.mdx} + \dfrac{M_{Fat}}{S_{x.i}} = 4.53 \cdot ksi$ *Fatigue Stress range in Member, away from connection*

$\sigma_{top.fat.mdx} = 1.77 \cdot ksi$ *Fatigue Stress Range in member near welded connections*

$Check_8 := if(\sigma_{top.fat} < F_{n.B} \wedge \sigma_{top.fat.mdx} < F_{n.E}, \text{"Member OK in Fatigue"}, \text{"Member No Good in Fatigue"})$

$Check_8 = \text{"Member OK in Fatigue"}$

- ***Check Bent Plate Capacity***

Check plate thickness to prevent prying action in accordance with AISC 13th edition, Chapter 9.

$$T := \frac{\max\left(\left|C_{top}\right|, T_{top}\right)\cdot\sin(\theta)}{4} = 3.86\,kip$$

Equivalent prying force, per bolt

$$b' := S_{pl.end} + 1\,in - \frac{d_{bolt}}{2} = 2.06\,in$$

Distance from bend in plate to edge of bolt hole

$$t_{min} := \sqrt{\frac{4.44\cdot T\cdot b'}{S_{t1}\cdot F_u}} = 0.43\,in$$

Minimum plate thickness required to avoid additional prying force in bolt

$$Check_{pry} := if\left(t_{min} < t_{gus}, \text{"No Bolt Prying will occur"}, \text{"Bolt prying will occur, Thicken Gusset Plate"}\right)$$

$$\boxed{Check_{pry} = \text{"No Bolt Prying will occur"}}$$

Check Horizontal Bending Stress in Plate

$$M_{pl} := 4\cdot T\cdot\left(b' + \frac{d_{bolt}}{2} + 0.5\cdot S_{g.end}\right) = 50.23\cdot kip\cdot in$$

Bending Moment in Gusset plate - assume point of fixity is half way between bolts and tip of gusset in compression

$$M_r := F_y\cdot\frac{h_{gus.t}\cdot t_{gus}^{2}}{6} = 56.25\cdot kip\cdot in$$

Moment capacity of plate, assuming section can reach first yield in section under consideration

$$Check_{pl} := if\left(M_r > M_{pl}, \text{"Bending in Gusset Plate Tip OK"}, \text{"Bending in Gusset Plate Tip No Good - Thicken Plate"}\right)$$

$$\boxed{Check_{pl} = \text{"Bending in Gusset Plate Tip OK"}}$$

Check Vertical Bending in Plate

$V_{top.str} := V_{u.Str} = 18.99\,kip$ *Vertical shear*

$L_b := \frac{(6in + 1in + S_{g.end})}{\cos(skew)^2} = 17\,in$ *Cantilever length and unbraced length of gusset from end of gusset to bolts, increased by skew to help account for out-of-plane effects*

$M_{pl.v} := V_{top.str} \cdot (L_b) = 322.8 \cdot kip \cdot in$ *Vertical Bending moment in plate due to gravity loads*

$C_b := 1.0$ *Lateral Torsional Buckling coeeficietn (AISC F1-1) - conservatively take as 1.0*

Moment Capacity per AISC Specifications Section F11:

$$M_p := F_y \cdot 1.5 \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6} = 1.35 \times 10^3 \cdot kip \cdot in$$

$$M_n := \begin{cases} M_p & \text{if } \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} < \frac{0.08E_s}{F_y} \\ \min\left[M_p, C_b \cdot \left[1.52 - 0.274 \cdot \left(\frac{L_b \cdot h_{gus.t}}{t_{gus}^2}\right) \cdot \frac{F_y}{E_s}\right] \cdot \left(F_y \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6}\right)\right] & \text{if } \left(\frac{0.08E_s}{F_y} < \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} \wedge \frac{L_b \cdot h_{gus.t}}{t_{gus}^2} < \frac{1.9E_s}{F_y}\right) \\ \min\left(M_p, \frac{1.9 \cdot E_s \cdot C_b}{\frac{L_b \cdot h_{gus.t}}{t_{gus}^2}} \cdot \frac{t_{gus} \cdot h_{gus.t}^2}{6}\right) & \text{otherwise} \end{cases}$$

$M_n = 1.21 \times 10^3 \cdot kip \cdot in$ *Moment Capacity of plate (reduction factor is 1.0)*

$Check_{pl.v} := \text{if}(M_n > M_{pl.v}, \text{"Vertical Bending in Plate OK"}, \text{"Vertical Bending No Good, Thicken Plate"})$

$Check_{pl.v} = \text{"Vertical Bending in Plate OK"}$

- ***Connection Capacity Checks***

- *Block Shear Rupture*

$\phi_{bs} := 0.80$ *Resistance Factor for Block Shear*

$t := \min(t_{gus}, t_{pl}) = 0.75\,in$ *Minimum plate thickness for rupture checks*

- *Member Capacity - Option 1*

$A_{vg} := 2 \cdot S_{t.end} \cdot t = 2.25\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := 2 \cdot (S_{t.end} - 0.5d_{h_tens}) \cdot t = 1.55\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (2S_{t1} + S_{t2}) \cdot t = 6.75\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := (2S_{t1} + S_{t2} - 3d_{h_tens}) \cdot t = 4.64\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.i1} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 287.97\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

- *Member Capacity - Option 2*

$A_{vg} := 4 \cdot S_{t.end} \cdot t = 4.5\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := 4 \cdot (S_{t.end} - 0.5d_{h_tens}) \cdot t = 3.09\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (2S_{t1}) \cdot t = 4.5\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := (2S_{t1} - 2d_{h_tens}) \cdot t = 3.09\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.i2} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 254.18\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

$Check := if(\min(R_{r.i1}, R_{r.i2}) > T_{top}, \text{"Horiz Block Rupture OK"}, \text{"Horiz Block Rupture No Good"})$

Check = "Horiz Block Rupture OK"

- *Vertical Block Rupture in Gusset*

$V_{top.str} = 18.99\,kip$ *Vertical shear*

$A_{vg} := (S_{t.end} + 2 \cdot S_{t1} + S_{t2}) \cdot t = 7.87\,in^2$ *Gross Area Along Plane Resisting Shear*

$A_{vn} := A_{vg} - 3.5d_{h_tens} \cdot t = 5.41\,in^2$ *Net Area Along Plane Resisting Shear*

$A_{tg} := (S_{t.end}) \cdot t = 1.12\,in^2$ *Gross Area - Plane Resisting Tension*

$A_{tn} := A_{tg} - 0.5d_{h_tens} \cdot t = 0.77\,in^2$ *Net Area - Plane Resisting Tension*

$R_{r.v} := \phi_{bs} \cdot \min[(0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn}), (0.58 \cdot F_u \cdot A_{vn} + F_u \cdot A_{tn})] = 203.51\,kip$ *Block Shear Rupture Resistance [AASHTO 6.13.4-1]*

$Check := if(R_{r.v} > V_{top.str}, \text{"Vertical Block Rupture OK"}, \text{"Veritcal Block Rupture No Good"})$

Check = "Vertical Block Rupture OK"

- ***Bolt Forces***

$$I_p := 2\left[\left(\frac{S_{t2}}{2}\right)^2 + \left(\frac{S_{t2}}{2} + S_{t1}\right)^2\right] \qquad I_p = 45\,in^2$$

Bolt Pattern Inertia

$$y_{max} := S_{t1} + \frac{S_{t2}}{2} \qquad y_{max} = 4.5\,in$$

Distance from Bolt Group CG to outer Bolt

- *Strength I*

$$V_{top.str} = 18.99\,kip$$

Shear Acting on Bolts due to LL+IM

$$M_{t.bolt.str} := V_{top.str}\cdot\left(5in + S_{t.end}\right) = 10.29\cdot kip\cdot ft$$

Moment Acting on Bolts due to eccentricity

$$F_{m.str} := \frac{M_{t.bolt.str}\cdot y_{max}}{I_p} = 12.34\,kip$$

Force on Outer Bolt Due to moment

$$V_{bolt.str} := \sqrt{\left(F_{m.str} + \frac{\max\left(\left|C_{top}\right|, T_{top}\right)}{N_{bolts}}\right)^2 + \left(\frac{V_{top.str}}{N_{bolts}}\right)^2} = 19.45\,kip$$

Ultimate Shear Force per Bolt, Strength I

- *Service II*

$$V_{top.serv} := V_{u.Ser2} = 14.17\,kip$$

Shear Acting on Bolts due to LL+IM

$$M_{t.bolt.serv} := V_{top.serv}\cdot\left(5in + S_{t.end}\right) = 7.68\cdot kip\cdot ft$$

Moment Acting on Bolts due to eccentricity

$$F_{m.serv} := \frac{M_{t.bolt.serv}\cdot y_{max}}{I_p} = 9.21\,kip$$

Force on Outer Bolt Due to moment

$$V_{bolt.serv} := \sqrt{\left(F_{m.serv} + \frac{F_{top.mdx}}{N_{bolts}}\right)^2 + \left(\frac{V_{top.serv}}{N_{bolts}}\right)^2} = 14.48\,kip$$

Ultimate Shear Force per Bolt, Service II

- *Fatigue*

$V_{top.fat} := V_{u.fat} = 12.52\,kip$ *Shear Acting on Bolts due to LL+IM*

$M_{t.bolt.fat} := V_{top.fat} \cdot (S_{t.end} + 5in) = 6.78 \cdot kip \cdot ft$ *Moment Acting on Bolts due to eccentricity*

$$\sigma_{top.fat} := \frac{M_{t.bolt.fat} \cdot (S_{t1} + 0.5S_{t2})}{\frac{t_{gus} \cdot h_{gus.t}^{3}}{12}} + \sigma_{top.fat.mdx} = 5.16 \cdot ksi$$

Fatigue Stress Acting in plate at Extreme Bolt Location

- ***Bolt Bearing on Material***

$t_{mat} := \min(t_{pl}, t_{w.i}) = 0.43\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min[(S_{t1} - d_{h_stndrd}), (S_{t2} - d_{h_stndrd}), (S_{t.end} - 0.5d_{h_stndrd})] = 1.03\,in$

Minimum of Clear Distance Between Holes Or Edge of Material in the Direction of Loading [6.13.2.9]

$$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases} = 34.35\,kip$$

Nominal Resistance in Bearing, Per Bolt

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 27.48\,kip$ *Resistance of Single Bolt in Bearing*

$Check_9 := \text{if}(|V_{bolt.str}| \le \phi_{bb} \cdot R_n, \text{"Top Chord Bolt Bearing OK"}, \text{"Top Chord Bolt Bearing No Good"})$

$Check_9 = \text{"Top Chord Bolt Bearing OK"}$

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [AASHTO 6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Factored Resistance Per Bolt*

$Check_{10} := \text{if}(V_{bolt.str} \le \phi_s \cdot F_{v_bolt}, \text{"Top Chord Bolt Shear OK"}, \text{"Top Chord Bolt Shear No Good"})$

$Check_{10} = \text{"Top Chord Bolt Shear OK"}$

- ***Check Slip Resistance Under Service II***

$P_{slip.top} := V_{bolt.serv} = 14.48\,kip$

$R_{n.slip.top} := K_h \cdot K_s \cdot N_s \cdot P_t = 19.5\,kip$ *Slip Resistance Per Bolt [AASHTO 6.13.2.8-1]*

$Check_{11} := if\left(P_{slip.top} < R_{n.slip.top}, \text{"Top Chord OK in Slip Resistance"}, \text{"Top Chord No Good in Slip Resistance"}\right)$

$Check_{11} = \text{"Top Chord OK in Slip Resistance"}$

- ***Check Fatigue at Bolts***

$\sigma_{top.fat} = 5.16 \cdot ksi$ *Fatigue Stress Range*

$F_{n.B} = 9.99 \cdot ksi$ *Fatigue Thershold [AASHTO Table 6.6.1.2.5-3, Category B]*

$Check_{12} := if\left(F_{n.B} > \sigma_{top.fat}, \text{"Top Chord Connection OK in Fatigue"}, \text{"Top Chord Connection No Good in Fatigue"}\right)$

$Check_{12} = \text{"Top Chord Connection OK in Fatigue"}$

Diagonal Chord Design Check:

- ***Member Forces***

$C_{diag} := \max\left(\left|C_{diag.mdx}\right|, P_{diag.str}, \left|1.4F_{diag}\right|\right) = 39.36\,kip$

Total Compression Force in Diagonal Member, max of Strength I or Strength 3

$T_{diag} := T_{diag.mdx} = 33.58\,kip$

Total Tension Force in Diagonal Member, Strength I

$F_{diag} := \max\left(\left|F_{diag.mdx}\right|, \left|P_{diag.ser2}\right|\right) = 29.4\,kip$

Max Force in Diagonal Member, Service II

$\sigma_{diag.fat} := \max\left(\sigma_{diag.fat.mdx}, \left|\frac{P_{diag.fat}}{A_{ang}}\right|\right) = 3\cdot ksi$

Fatigue Stress range in Member

- ***Member Capacity***
  - ***Compression***

$KLr_{eff} := \text{if}\left(\frac{L_{dg}}{r_{xy.ang}} \le 80, 72 + 0.75\cdot\frac{L_{dg}}{r_{xy.ang}}, 32 + 1.25\frac{L_{dg}}{r_{xy.ang}}\right) = 110.81$

Effective KL/r (AASHTO EQ 6.9.4.4-1 & 2), allows to only check flexural buckling failure mechanism - no LTB or combined force check needed

$Q := \begin{cases} 1 & \text{if } \frac{b_{f.ang}}{t_{f.ang}} \le 0.45\cdot\sqrt{\frac{E_s}{F_y}} \\ \text{"Slender"} & \text{otherwise} \end{cases} = 1$

Slender Element Reduction Factor [AASHTO 6.9.4.2.1-1]

$P_o := Q\cdot F_y\cdot A_i = 335\cdot kip$

Equivalent nominal yield resisitance [AASHTO 6.9.4.1.1]

$P_{e.ang} := \frac{\pi^2\cdot E_s}{\left(KLr_{eff}\right)^2}\cdot A_{ang} = 87.42\cdot kip$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

$P_{n.ang} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.ang}}}\cdot P_o\right) & \text{if } \frac{P_{e.ang}}{P_o} \ge 0.44 \\ \left(0.877\cdot P_{e.ang}\right) & \text{otherwise} \end{cases} = 76.67\cdot kip$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c\cdot P_{n.ang} = 72.83\,kip$

Factored Compressive Resistance of Angle

$Check_{13} := \text{if}\left(\phi_c\cdot P_{n.ang} \ge \left|C_{diag}\right|, \text{"Diagonal Chord OK in Compression"}, \text{"Diagonal Chord No Good in Compression"}\right)$

$Check_{13} = \text{"Diagonal Chord OK in Compression"}$

  - ***Tension***

$\phi P_{ny.ang} := \phi_y\cdot F_y\cdot A_{ang} = 178.12\,kip$

Factored Tensile Resistance in Yielding [6.8.2.1-1]

$U := 0.50$

Reduction Factor [6.8.2.2]

$\phi P_{nu} := \phi_u\cdot F_u\cdot A_{ang}\cdot U = 97.5\,kip$

Factored Tensile Resistance in Fracture [6.8.2.1-2]

$Check_{14} := \text{if}\left(\min\left(\phi P_{ny.ang}, \phi P_{nu}\right) \ge \left|T_{diag}\right|, \text{"Diagonal Chord OK in Tension"}, \text{"Diagonal Chord No Good in Tension"}\right)$

$Check_{14} = \text{"Diagonal Chord OK in Tension"}$

- ***Determine Weld Length Along Member***

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld (6.5.4.2)*

$\sigma_{diag.fat} = 3 \cdot ksi$ *Fatigue Stress Range*

$$L_{fat.req.w} := \frac{\sigma_{diag.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} = 5.1\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req} := \frac{\max(C_{diag}, T_{diag})}{\phi_{e2} \cdot 0.60F_{exx} \cdot 0.221in} = 5.3\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Weld_{diag} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$$

$$\boxed{Weld_{diag} = 10\,in}$$

- ***Determine Top Gusset Plate - to - Top Chord Weld***

Allowable Fatigue Stress based on equations 6.6.1.2.5-4:

$$L_{fat.req.t} := \frac{2\sigma_{diag.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} \qquad L_{fat.req.t} = 10.21\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req.t} := \frac{2\max(C_{diag}, T_{diag})}{\phi_u \cdot 0.60F_u \cdot 0.221in} \qquad L_{Str.req.t} = 11.42\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Req_Weld_Length_{tgus} := \max(L_{fat.req.t}, L_{Str.req.t})$$

$$\boxed{Req_Weld_Length_{tgus} = 11.42\,in}$$

- *Check Member Fatigue*

$$\sigma_{diag.fat.e} := \frac{\sigma_{diag.fat}}{1 - \dfrac{y_{o.ang}}{\max\left(b_{f.ang}, \dfrac{Weld_{diag} - b_{f.ang}}{2}\right)}} = 4.26 \cdot ksi$$

Effective fatigue stress on member (AASHTO Table 6.6.1.2.3-1, section 7.2)

$$Check := \text{if}\left(F_{n.E} > \sigma_{diag.fat.e}, \text{"Diagonal Chord Weld OK in Fatigue"}, \text{"Diaganol Chord Weld No Good in Fatigue"}\right)$$

$$\boxed{Check = \text{"Diagonal Chord Weld OK in Fatigue"}}$$

Bottom Chord Design Check:

- ***Member Forces***

$C_{bot} := \max\left(\left|C_{bot.mdx}\right|, \left|1.4 \cdot F_{bot.chord}\right|\right) = 13.09\,kip$

Total Compression Force in Bottom Member, max of Strength I or Strength III

$T_{bot} := \max\left(T_{bot.mdx}, P_{diag.str} \cdot \cos(\theta)\right) = 31.7\,kip$

Total Tension Force in Bottom Member, Strength I

$F_{bot} := \max\left(\left|F_{bot.mdx}\right|, \left|P_{diag.ser2}\right| \cdot \cos(\theta)\right) = 23.68\,kip$

Max Force in Bottom Member, Service II

$\sigma_{bot.fat} := \max\left(\sigma_{bot.fat.mdx}, \left|\frac{P_{diag.fat}}{A_{ang}}\right| \cdot \cos(\theta)\right) = 2.42 \cdot ksi$

Fatigue Stress range in Member

- ***Member Capacity***
  - ***Compression***

$KLr_{eff} := \text{if}\left(\frac{L_h}{r_{xy.ang}} \le 80, 72 + 0.75 \cdot \frac{L_h}{r_{xy.ang}}, 32 + 1.25 \frac{L_h}{r_{xy.ang}}\right) = 144.38$

Effective KL/r (AASHTO EQ 6.9.4.4-1 & 2), allows to only check flexural buckling failure mechanism - no LTB or combined force check needed

$P_{e.ang} := \frac{\pi^2 \cdot E_s}{\left(KLr_{eff}\right)^2} \cdot A_{ang} = 51.49 \cdot kip$

Elastic buckling capacity based on Flxural Buckling Failure (AASHTO 6.9.4.1.2-1)

$$P_{n.ang} := \begin{cases} \left(0.658^{\frac{P_o}{P_{e.ang}}} \cdot P_o\right) & \text{if } \frac{P_{e.ang}}{P_o} \ge 0.44 \\ \left(0.877 \cdot P_{e.ang}\right) & \text{otherwise} \end{cases} = 45.16 \cdot kip$$

Nominal compressive resistance in bottom portion of column [AASHTO 6.9.4.1.1-1 & 6.9.4.1.1-2]

$\phi_c \cdot P_{n.ang} = 42.9\,kip$

Factored Compressive Resistance of Angle

$Check_{15} := \text{if}\left(\phi_c \cdot P_{n.ang} \ge \left|C_{bot}\right|, \text{"Bottom Chord OK in Compression"}, \text{"Bottom Chord No Good in Compression"}\right)$

$Check_{15} = \text{"Bottom Chord OK in Compression"}$

  - ***Tension***

$\phi P_{ny.ang} := \phi_y \cdot F_y \cdot A_{ang} = 178.12\,kip$

Factored Tensile Resistance in Yielding [6.8.2.1-1]

$U := 0.50$

Reduction Factor [6.8.2.2]

$\phi P_{nu} := \phi_u \cdot F_u \cdot A_{ang} \cdot U = 97.5\,kip$

Factored Tensile Resistance in Fracture [6.8.2.1-2]

$Check_{16} := \text{if}\left(\min\left(\phi P_{ny.ang}, \phi P_{nu}\right) \ge \left|T_{bot}\right|, \text{"Bottom Chord OK in Tension"}, \text{"Bottom Chord No Good in Tension"}\right)$

$Check_{16} = \text{"Bottom Chord OK in Tension"}$

- *Determine Weld Length Along Member*

$\phi_{e2} := 0.80$ *Resistance of Fillet Weld*

$\sigma_{bot.fat} = 2.42 \cdot ksi$ *Fatigue Stress Range*

$$L_{fat.req.w} := \frac{\sigma_{bot.fat} \cdot A_{ang}}{F_{n.B} \cdot 0.221in} = 4.11\,in$$

Required Length of Weld, Based on Fatigue and assuming a 5/16" weld size

$$L_{Str.req} := \frac{\max(C_{bot}, T_{bot})}{\phi_u \cdot 0.60F_u \cdot 0.221in} = 4.6\,in$$

Required Length of Weld, Based on Strength I and assuming a 5/16" weld size

$$Weld_{bot} := \max(L_{fat.req.w}, L_{Str.req}, 10in)$$

$$\boxed{Weld_{bot} = 10\,in}$$

- *Check Member Fatigue*

$$\sigma_{bot.fat.e} := \frac{\sigma_{bot.fat}}{1 - \frac{y_{o.ang}}{\max\left(b_{f.ang}, \frac{Weld_{bot} - b_{f.ang}}{2}\right)}} = 3.43 \cdot ksi$$

Effective fatigue stress on member (AASHTO Table 6.6.1.2.3-1, section 7.2)

$$Check := \text{if}\left(F_{n.E} > \sigma_{bot.fat.e}, \text{"Bottom Chord Weld OK in Fatigue"}, \text{"Bottom Chord Weld No Good in Fatigue"}\right)$$

$$\boxed{Check = \text{"Bottom Chord Weld OK in Fatigue"}}$$

Check Bottom Gusset Plates:

Conservatively ignore skew for checking shear in bolts

$N_{bolts} := 5$

$I_p := \frac{N_{bolts}}{12} \cdot \left[S_{g.row}^2 \cdot \left(N_{bolts}^2 - 1 \right) \right]$ $\quad I_p = 90\,in^2$ *Bolt Pattern Inertia*

$ecc := 1.5 \cdot S_{g.row} = 4.5\,in$ *Eccentricity of Force on Bolt Group*

$y_{max} := \frac{S_{g.row}}{2} \cdot \left(N_{bolts} - 1 \right)$ $\quad y_{max} = 6\,in$ *Distance from Bolt Group CG to outer Bolt*

- ***Strength I: Compression in Diagonals, Tension in Bottom Chord***

$F_{x.bolts} := \left(C_{diag} \cdot \cos(\theta) - T_{bot} \right) \div N_{bolts} = 0\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := C_{diag} \cdot \sin(\theta) \div N_{bolts} = 4.66\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \frac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 0\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts} \right)^2 + F_{y.bolts}^2} = 4.66\,kip$ *Total Shear Force on Outer Bolt*

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Nominal Resistance Per Bolt*

$Check_{17} := if\left(V_{bolts} < \phi_s \cdot F_{v_bolt}, \text{"Bot Gusset Bolt Shear OK"}, \text{"Bot Gusset Bolt Shear No Good"} \right)$

$Check_{17} = \text{"Bot Gusset Bolt Shear OK"}$

- ***Bolt Bearing on Material***

$t_{mat} := \min\left(t_{pl}, t_{gus} \right)$ $\quad t_{mat} = 0.75\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min\left[\left(S_{g.row} - d_{h_stndrd} \right), \left(S_{pl.end} - 0.5 d_{h_stndrd} \right), \left(S_{g.end} - 0.5 d_{h_stndrd} \right) \right]$

$L_c = 1.03\,in$ *Minimum of Clear Distance Between Holes O Edge of Material in the Direction of Loading [6.13.2.9]*

$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2 d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases}$ $\quad R_n = 60.33\,kip$ *Nominal Resistance in Bearing, Per Bolt*

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 48.26\,kip$ *Resistance of Single Bolt in Bearing*

$Check_{18} := if\left(V_{bolts} < \phi_{bb} \cdot R_n, \text{"Bot Gusset Bolt Bearing OK"}, \text{"Bot Gusset Bolt Bearing No Good"} \right)$

$Check_{18} = \text{"Bot Gusset Bolt Bearing OK"}$

- ***Service II: Forces due to Axle Loads Included***

$F_{x.bolts} := \left(F_{diag} \cdot \cos(\theta) - F_{bot}\right) \div N_{bolts} = 0\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := F_{diag} \cdot \sin(\theta) \div N_{bolts} = 3.48\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \dfrac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 0\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\ 2}} = 3.48\,kip$

Total Shear Force on Outer Bolt

- ***Check Slip Resistance Under Service II***

$R_{n.slip.bot} := K_h \cdot K_s \cdot N_s \cdot P_t$ $R_{n.slip.bot} = 19.5\,kip$ *Nominal Slip REsistance per Bolt [AASHTO 6.13.2.8-1]*

$Check_{19} := \text{if}\left(V_{bolts} < R_{n.slip.bot}, \text{"Bot Gusset OK in Slip Resistance"}, \text{"Bot Gusset No Good in Slip Resistance"}\right)$

$Check_{19} = \text{"Bot Gusset OK in Slip Resistance"}$

- ***Fatigue: Forces due to Axle Loads Included***

$\sigma_{x.gus} := \left(\sigma_{diag.fat} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat} \cdot A_{ang}\right) \div \left(t_{gus} \cdot h_{gus.b}\right)$

$\sigma_{x.gus} = 0 \cdot ksi$ *Horizontal Force Acting on Group*

$\sigma_{m.gus} := \dfrac{\left(\sigma_{diag.fat} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat} \cdot A_{ang}\right) \cdot ecc \cdot y_{max}}{\dfrac{h_{gus.b}^{\ 3} \cdot t_{gus}}{12}}$ $\sigma_{m.gus} = 0 \cdot ksi$ *Stress in Extreme Fiber of Gusset Plate due to Eccentric Forces*

$\sigma_{gus.max} := \left|\sigma_{x.gus}\right| + \left|\sigma_{m.gus}\right|$ $\sigma_{gus.max} = 0 \cdot ksi$ *Max Stress in Plate*

$F_{th} := 16ksi$ *Allowable Fatigue Stress, Category B, [Table 6.6.1.2.5-3]*

$Check_{20} := \text{if}\left(F_{th} > \sigma_{gus.max}, \text{"Bot Gusset Connection OK in Fatigue"}, \text{"Bot Gusset Connection No Good in Fatigue"}\right)$

$Check_{20} = \text{"Bot Gusset Connection OK in Fatigue"}$

- ***Strength I: Tension in Diagonals, Compression in Bottom Chord***

$F_{x.bolts} := \left(T_{diag} \cdot \cos(\theta) - C_{bot}\right) \div N_{bolts} = 2.79\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := T_{diag} \cdot \sin(\theta) \div N_{bolts} = 3.98\,kip$ *Vertical Force Acting on Group, per bolt*

$F_{mx.bolts} := \dfrac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p} = 4.19\,kip$ *Force Due to Ecentricity on Outer Bolt*

$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\ 2}} = 8.03\,kip$

Total Shear Force on Outer Bolt

- ***Check Bolt Shear Resistance***

$F_{v_bolt} := 0.38 \cdot A_b \cdot F_{ub}$ *Shear Strength of Single Bolt, Assuming 1 Shear Plane and Threads in the Shear Plane [6.13.2.7-2]*

$\phi_s := 0.80$ *Resistance Factor for Bolts in Shear [6.5.4.2]*

$\phi_s \cdot F_{v_bolt} = 21.94\,kip$ *Nominal Resistance Per Bolt*

$Check_{21} := \text{if}\left(V_{bolts} < \phi_s \cdot F_{v_bolt}, \text{"Bot Gusset Bolt Shear OK"}, \text{"Bot Gusset Bolt Shear No Good"}\right)$

$Check_{21} = \text{"Bot Gusset Bolt Shear OK"}$

- ***Bolt Bearing on Material***

$t_{mat} := \min\left(t_{pl}, t_{gus}\right)$ $t_{mat} = 0.75\,in$ *Minimum Thickness of Material in Bearing*

$L_c := \min\left[\left(S_{g.row} - d_{h_stndrd}\right), \left(S_{pl.end} - 0.5d_{h_stndrd}\right), \left(S_{g.end} - 0.5d_{h_stndrd}\right)\right]$

$L_c = 1.03\,in$ *Minimum of Clear Distance Between Holes O Edge of Material in the Direction of Loading [6.13.2.9]*

$R_n := \begin{cases} 2.4 \cdot d_{bolt} \cdot t_{mat} \cdot F_u & \text{if } L_c > 2d_{bolt} \\ 1.2 \cdot L_c \cdot t_{mat} \cdot F_u & \text{otherwise} \end{cases}$ $R_n = 60.33\,kip$ *Nominal Resistance in Bearing, Per Bolt*

$\phi_{bb} := 0.80$ *Resistance Factor for Bearing [6.5.4.2]*

$\phi_{bb} \cdot R_n = 48.26\,kip$ *Resistance of Single Bolt in Bearing*

$Check_{22} := \text{if}\left(V_{bolts} < \phi_{bb} \cdot R_n, \text{"Bot Gusset Bolt Bearing OK"}, \text{"Bot Gusset Bolt Bearing No Good"}\right)$

$Check_{22} = \text{"Bot Gusset Bolt Bearing OK"}$

- ***Service II: Without Forces Induced by Axle Loads***

$F_{x.bolts} := \left(F_{diag.mdx} \cdot \cos(\theta) - F_{bot.mdx}\right) \div N_{bolts}$ $\quad F_{x.bolts} = 2.61\,kip$ *Horizontal Force Acting on Group, per bolt*

$F_{y.bolts} := F_{diag.mdx} \cdot \sin(\theta) \div N_{bolts}$ $\quad F_{y.bolts} = 2.95\,kip$ *Vertical Force Acting on Group, per bolt*

$$F_{mx.bolts} := \frac{F_{x.bolts} \cdot N_{bolts} \cdot ecc \cdot y_{max}}{I_p}$$ $\quad F_{mx.bolts} = 3.92\,kip$ *Force Due to Ecentricity on Outer Bolt*

$$V_{bolts} := \sqrt{\left(F_{x.bolts} + F_{mx.bolts}\right)^2 + F_{y.bolts}^{\;2}}$$

$V_{bolts} = 7.17\,kip$ *Total Shear Force on Outer Bolt*

- ***Check Slip Resistance Under Service II***

$R_{n.slip.bot} := K_h \cdot K_s \cdot N_s \cdot P_t$ $\quad R_{n.slip.bot} = 19.5\,kip$ *Nominal Slip REsistance per Bolt [AASHTO 6.13.2.8-1]*

$Check_{23} := \text{if}\left(V_{bolts} < R_{n.slip.bot}, \text{"Bot Gusset OK in Slip Resistance"}, \text{"Bot Gusset No Good in Slip Resistance"}\right)$

$Check_{23} = \text{"Bot Gusset OK in Slip Resistance"}$

- ***Fatigue: Without Forces Induced by Axle Loads***

$\sigma_{x.gus} := \left(\sigma_{diag.fat.mdx} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat.mdx} \cdot A_{ang}\right) \div \left(t_{gus} \cdot h_{gus.b}\right)$

$\sigma_{x.gus} = 0.15 \cdot ksi$ *Horizontal Force Acting on Group*

$$\sigma_{m.gus} := \frac{\left(\sigma_{diag.fat.mdx} \cdot A_{ang} \cdot \cos(\theta) - \sigma_{bot.fat.mdx} \cdot A_{ang}\right) \cdot ecc \cdot y_{max}}{\frac{h_{gus.b}^{\;3} \cdot t_{gus}}{12}}$$ $\quad \sigma_{m.gus} = 0.15 \cdot ksi$ *Stress in Extreme Fiber of Gusset Plate due to Eccentric Forces*

$\sigma_{gus.max} := \left|\sigma_{x.gus}\right| + \left|\sigma_{m.gus}\right|$ $\quad \sigma_{gus.max} = 0.29 \cdot ksi$ *Max Stress in Plate*

$F_{th} := 16ksi$ *Allowable Fatigue Stress, Category B, [Table 6.6.1.2.5-3]*

$Check_{24} := \text{if}\left(F_{th} > \sigma_{gus.max}, \text{"Bot Gusset Connection OK in Fatigue"}, \text{"Bot Gusset Connection No Good in Fatigue"}\right)$

$Check_{24} = \text{"Bot Gusset Connection OK in Fatigue"}$

Summary:

$$Summary_Checks := \begin{pmatrix} Check_1 \\ Check_2 \\ Check_3 \\ Check_4 \\ Check_5 \\ Check_6 \\ Check_7 \\ Check_8 \\ Check_9 \\ Check_{10} \\ Check_{11} \\ Check_{12} \\ Check_{13} \\ Check_{14} \\ Check_{15} \\ Check_{16} \\ Check_{17} \\ Check_{18} \\ Check_{19} \\ Check_{20} \\ Check_{21} \\ Check_{22} \\ Check_{23} \\ Check_{24} \end{pmatrix}$$

$$Summary_Checks = \begin{pmatrix} \text{"Cross Frame Depth OK"} \\ \text{"Bottom Chord Slenderness No Good"} \\ \text{"Top Chord Slenderness OK"} \\ \text{"Diagonal Chord Slenderness OK"} \\ \text{"Angle Plate Slenderness OK"} \\ \text{"Top Chord OK in Comp/Bend"} \\ \text{"Top Chord OK in Tension"} \\ \text{"Member OK in Fatigue"} \\ \text{"Top Chord Bolt Bearing OK"} \\ \text{"Top Chord Bolt Shear OK"} \\ \text{"Top Chord OK in Slip Resistance"} \\ \text{"Top Chord Connection OK in Fatigue"} \\ \text{"Diagonal Chord OK in Compression"} \\ \text{"Diagonal Chord OK in Tension"} \\ \text{"Bottom Chord OK in Compression"} \\ \text{"Bottom Chord OK in Tension"} \\ \text{"Bot Gusset Bolt Shear OK"} \\ \text{"Bot Gusset Bolt Bearing OK"} \\ \text{"Bot Gusset OK in Slip Resistance"} \\ \text{"Bot Gusset Connection OK in Fatigue"} \\ \text{"Bot Gusset Bolt Shear OK"} \\ \text{"Bot Gusset Bolt Bearing OK"} \\ \text{"Bot Gusset OK in Slip Resistance"} \\ \text{"Bot Gusset Connection OK in Fatigue"} \end{pmatrix}$$

Additional Top Gusset Bent Plate Checks:

$Check_{pry}$ = "No Bolt Prying will occur"

$Check_{pl}$ = "Bending in Gusset Plate Tip OK"

$Check_{pl.v}$ = "Vertical Bending in Plate OK"

Check = "Bottom Chord Weld OK in Fatigue"

Check = "Bottom Chord Weld OK in Fatigue"

3c. Ancillary Items

Aspects of the bridge that may have been influenced due to the change in deck thickness and removed the wearing surface were also considered. These items include, but are not limited to, elastomeric bearing design, foundations, abutment stability and concrete design. It was found that the applied pressure to the elastomeric bearings increased approximately 3.5%. This is a localized and maximum force which would dissipate quickly in the overall abutment design due to concurrent live load force and self-weight considerations. Any changes in design forces due to the changed deck are assumed to be within assumption limitations and safety margin of design.

The following provides geometric changes to the bridge seats and an overview of the changed bearing pressure.

Existing Pedestal Elevations

Designer: T. Nguyen Date: 12/9/2015
Checker: T. Poulin Date: 12/10/2015

Profile Information

PVC :			
Station =	13+10.		
Elevation =	728.35		
PVI :		G1 =	2.2357%
Station =	14+50.		
Elevation =	731.48		
PVT :		G2 =	-2.4500%
Station =	15+90.		
Elevation =	728.05		
		L=	280
Cross Slope =	-2.0% (from centerline)		

Abutment 1 Pedestal Elevations

	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5
Station Location	13+41.	13+48.5	13+56.	13+63.5	13+71.
Station Offset (ft) =	-15.000	-7.500	0.000	7.500	15.000
Wearing Surface Elevation =	728.663	728.937	729.201	729.157	729.102
Pavement Thickness (in) =	3	3	3	3	3
Deck Thickness (in) =	8	8	8	8	8
Web Depth (in) =	60	60	60	60	60
Bottom Flange (in) =	1.25	1.25	1.25	1.25	1.25
Bearing Height (in) =	6.38	6.75	6.875	6.375	6.625
Pedestal Elevation =	721.75	722.00	722.25	722.25	722.17
Blocking Height (in) =	4.33	4.24	4.29	4.25	4.31

Abutment 2 Pedestal Elevations

	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5
Station Location	15+13.	15+20.5	15+28.	15+35.5	15+43.
Station Offset (ft) =	-15.000	-7.500	0.000	7.500	15.000
Wearing Surface Elevation =	729.140	729.199	729.247	728.987	728.717
Pavement Thickness (in) =	3	3	3	3	3
Deck Thickness (in) =	8	8	8	8	8
Web Depth (in) =	60	60	60	60	60
Bottom Flange (in) =	1.25	1.25	1.25	1.25	1.25
Bearing Height (in) =	6.25	6.75	6.5	6.375	6.25
Pedestal Elevation =	722.25	722.25	722.33	722.08	721.82
Blocking Height (in) =	4.18	4.38	4.26	4.26	4.26

Proposed Pedestal Elevations

Designer: T. Nguyen Date: 12/9/2015
Checker: T. Poulin Date: 12/10/2015

Profile Information

PVC :			
Station =	13+10.		
Elevation =	728.35		
PVI :		G1 =	2.2357%
Station =	14+50.		
Elevation =	731.48		
PVT :		G2 =	-2.4500%
Station =	15+90.		
Elevation =	728.05		
		L=	280
Cross Slope =	-2.0% (from centerline)		

Abutment 1 Pedestal Elevations

	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5
Station Location	13+41.	13+48.5	13+56.	13+63.5	13+71.
Station Offset (ft) =	-15.000	-7.500	0.000	7.500	15.000
Wearing Surface Elevation =	728.663	728.937	729.201	729.157	729.102
Pavement Thickness (in) =	0.5	0.5	0.5	0.5	0.5
Deck Thickness (in) =	8.5	8.5	8.5	8.5	8.5
Blocking Height (in) =	4.33	4.24	4.29	4.25	4.31
Web Depth (in) =	60	60	60	60	60
Bottom Flange (in) =	1.25	1.25	1.25	1.25	1.25
Bearing Height (in) =	6.38	6.75	6.875	6.375	6.625
Pedestal Elevation =	721.92	722.17	722.42	722.42	722.34

Abutment 2 Pedestal Elevations

	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5
Station Location	15+13.	15+20.5	15+28.	15+35.5	15+43.
Station Offset (ft) =	-15.000	-7.500	0.000	7.500	15.000
Wearing Surface Elevation =	729.140	729.199	729.247	728.987	728.717
Pavement Thickness (in) =	0.5	0.5	0.5	0.5	0.5
Deck Thickness (in) =	8.5	8.5	8.5	8.5	8.5
Blocking Height (in) =	4.18	4.38	4.26	4.26	4.26
Web Depth (in) =	60	60	60	60	60
Bottom Flange (in) =	1.25	1.25	1.25	1.25	1.25
Bearing Height (in) =	6.25	6.75	6.5	6.375	6.25
Pedestal Elevation =	722.42	722.42	722.50	722.25	721.99

Proposed Backwall Elevation

Designer: T. Nguyen Date: 12/9/2015
Checker: T. Poulin Date: 12/10/2015

Profile Information

PVC :			
Station =	13+10.		
Elevation =	728.35		
PVI :		G1 =	2.2357%
Station =	14+50.		
Elevation =	731.48		
PVT :		G2 =	-2.4500%
Station =	15+90.		
Elevation =	728.05		
		L=	280
Cross Slope =	-2.0% (from centerline)		

Abutment 1 Backwall Elevations

	Gutterline near G1	Girder 3	Gutterline near G5
Station Location	13+35.14	13+52.64	13+70.14
Station Offset (ft) =	-17.500	0.000	17.500
Wearing Surface Elevation =	728.509	729.151	729.042
Pavement Thickness (in) =	0.5	0.5	0.5
Deck Thickness (in) =	8.5	8.5	8.5
Closed Cell Foam (in)	0.50	0.50	0.50
Backwall Elevation =	727.72	728.36	728.25

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Project:	Waitsfield	Job No.	411847.00	Sheet:	1 of 1
Item:	Bearing Designer	Designer:	TJP	Date:	12/11/15
		Checker:	JKO	Date:	12/11/15
				Grid:	1/10"

Existing Design Loads - per McFarland Johnson Design Plans:

Design Dead Load (unfactored) = 140^k

Design Live Load (unfactored) = 140^k

Design Longitudinal Movement = 1.57 in (Abutment 2)

Maximum Bearing stress = 1100 psi

Increase of load due to design changes: Increase Concrete deck thickness by 1"

$$= 1'' \cdot 7'6'' \cdot \left(\frac{172'}{2} + \frac{1'1''}{\cos(45°)}\right) \cdot 0.150^{k/ft^3}$$

$$= 8.2^k$$

New Design Dead Load (unfactored) = 148.2 Say $\underline{148^k}$

New Bearing Stress = $\dfrac{148^k + 140^k}{(9'')^2 \pi} = \underline{1131^k}$

The new bearing stress increases by < 3%. Considering construction tolerance's and safety factors, change of design not warranted.