CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357
PH. (207) 737-2007 FAX (207) 737-2008

Project: Craftsbury, Vt - Lifting and shipping Calculations for PBU's
Contractor: CCS Constructors
Client: JP Carrara & Sons
Engineering Design: Calderwood Engineering
Design Computations by: Eric T. Calderwood, PE

Project Notes:
Calculation of Centroid

Girders are straight - bridge is curved, radius is applied to bridge by varying the overhang, worst case condition will be assumed with the maximum overhang calculated for weight purposes, but the minimum overhange calculated for composite properties, and action

$t_{deck} := 8.5\ in$

$t_{deckOH} := 1\ ft + 4\ in = 1.33\ ft$ for fascias only on PBU 1 & 3

$\gamma_c := 150\ pcf$

$Curb_{Width} := 11\ in$ *Concrete Shear key/ pavement backer width*

$\gamma_{stl} := 490\ pcf$

$Curb_{Height} := 2.875\ in$ *Concrete Shear key/ pavement backer height*

Girder Properties:

Beam geometry/ Area/ weight

$b_{bf} := 16.0 \text{ in}$

$t_{bf} := 1 \text{ in}$ $\qquad A_{bf} := b_{bf} \cdot t_{bf} = 16 \text{ in}^2$

$d_w := 36 \text{ in}$

$t_w := 0.625 \text{ in}$ $\qquad A_w := d_w \cdot t_w = 22.5 \text{ in}^2$

$b_{tf} := 16.0 \text{ in}$

$t_{tf} := 1 \text{ in}$ $\qquad A_{tf} := b_{tf} \cdot t_{tf} = 16 \text{ in}^2$

$d_{beam} := t_{tf} + d_w + t_{bf} = 38 \text{ in}$

$A_g := b_{tf} \cdot t_{tf} + b_{bf} \cdot t_{bf} + d_w \cdot t_w = 54.5 \text{ in}^2$

$W_{stl} := A_g \cdot \gamma_{stl} \cdot 115\% \cdot 85.8612 \text{ ft} = 18311.54 \text{ lbf}$

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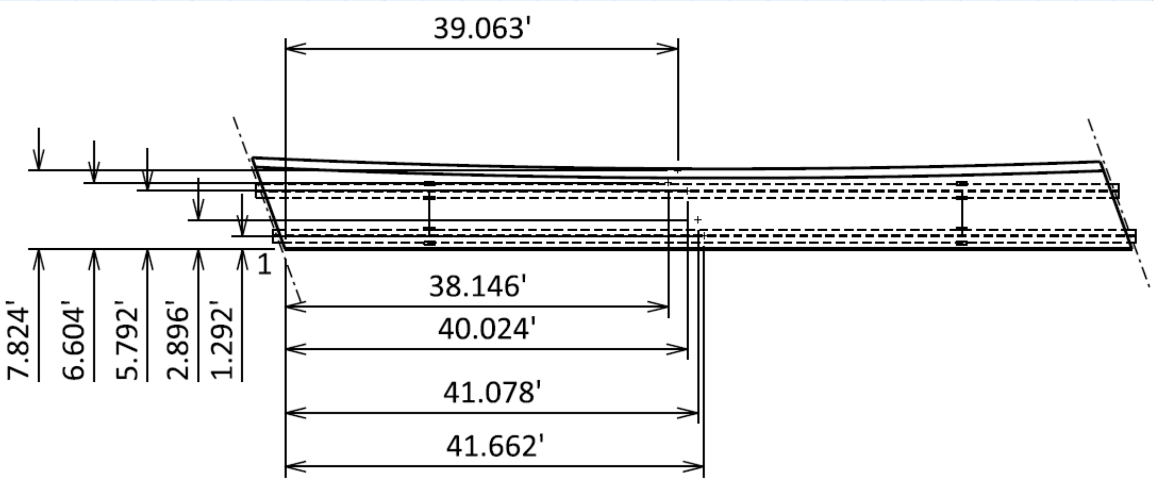
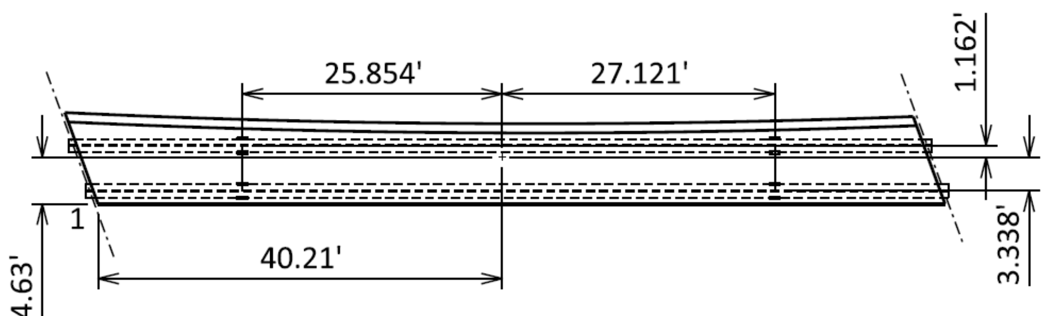
Centroid of PBU #1 (left Fascia) - x & y dimensions are measured from the lower left hand obtuse corner of the precast decking (abutment #1 end, Easterly side)



PBU #1



PBU #1

$\text{Haunch1} := 84.26\ \text{ft} \cdot b_{tf} \cdot 1.5\ \text{in} \cdot \gamma_c \cdot (1 + 0.5) = 3160\ \text{lbf}$

Half of the fascia beam haunch is calculated/measured with the overhang for PBU #1

$\text{Haunch2} := 84.26\ \text{ft} \cdot b_{tf} \cdot 1.5\ \text{in} \cdot \gamma_c \cdot (2) = 4213\ \text{lbf}$

Haunch for PBU #2

$\text{Haunch3} := \text{Haunch1} = 3160\ \text{lbf}$

Half of the fascia beam haunch is calculated/measured with the overhang for PBU #3

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$Steel1x := 41.662\ ft$ $Steel1y := 1.292\ ft$

$Steel2x := 40.024\ ft$ $Steel2y := 5.792\ ft$

$Conc1Area := 132.4988\ ft^2$ *concrete area as measured in cadd*

$Tc1 := t_{deckOH} + Curb_{Height} = 1.57\ ft$

$W_{c1} := Conc1Area \cdot Tc1 \cdot \gamma_c = 31261.44\ lbf$

$C1x := 39.063\ ft$ $C1y := 7.824\ ft$

$Conc2Area := 77.3941\ ft^2$ *concrete area as measured in cadd*

$Tc2 := t_{deckOH} = 1.33\ ft$

$W_{c2} := Conc2Area \cdot Tc2 \cdot \gamma_c = 15478.82\ lbf$

$C2x := 38.146\ ft$ $C2y := 6.604\ ft$

$Conc3Area := 488.0345\ ft^2$ *concrete area as measured in cadd*

$Tc3 := t_{deck} = 0.71\ ft$

$W_{c3} := Conc3Area \cdot Tc3 \cdot \gamma_c = 51853.67\ lbf$

$C3x := 41.078\ ft$ $C3y := 2.896\ ft$

$PBU1_{wt} := W_{stl} \cdot 2 + W_{c1} + W_{c2} + W_{c3} = 135217\ lbf$

$$PBU1_X := \frac{(Steel1x + Steel2x) \cdot W_{stl} + C1x \cdot W_{c1} + C2x \cdot W_{c2} + C3x \cdot W_{c3}}{PBU1_{wt}} = 40.21\ ft$$

$$PBU1_Y := \frac{(Steel1y + Steel2y) \cdot W_{stl} + C1y \cdot W_{c1} + C2y \cdot W_{c2} + C3y \cdot W_{c3}}{PBU1_{wt}} = 4.63\ ft$$

Haunch negligible in centroid calculations

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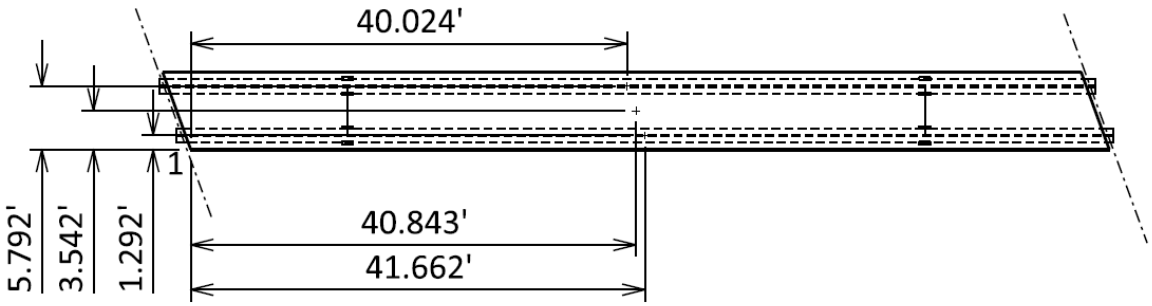
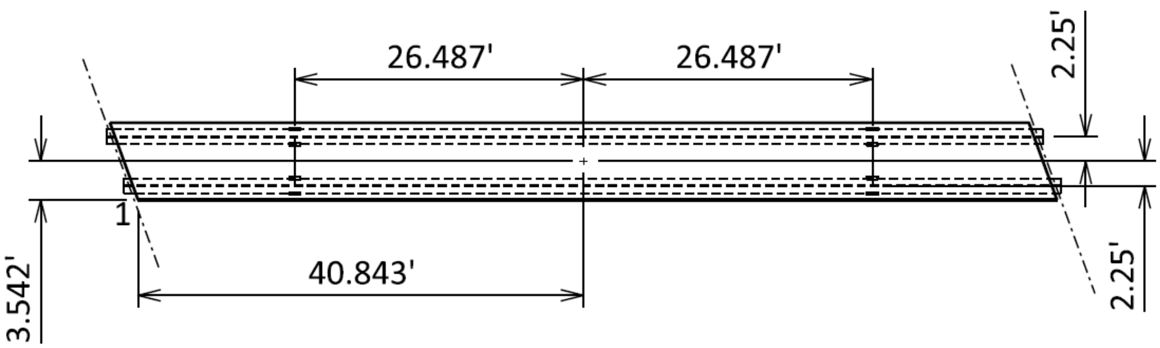
Centroid of PBU #2 (left Fascia) - x & y dimensions are measured from the lower left hand obtuse corner of the precast decking (abutment #1 end, Easterly side)



PBU #2



PBU #2

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$Steel1x := 41.662\ ft$ $\quad$ $Steel1y := 1.292\ ft$

$Steel2x := 40.024\ ft$ $\quad$ $Steel2y := 5.792\ ft$

$Conc1Area := 0\ ft^2$ *concrete area as measured in cadd*

$Tc1 := t_{deckOH} + Curb_{Height} = 1.57\ ft$

$W_{c1} := Conc1Area \cdot Tc1 \cdot \gamma_c = 0\ lbf$

$C1x := 0\ ft$ $\quad$ $C1y := 0\ ft$

$Conc2Area := 0\ ft^2$ *concrete area as measured in cadd*

$Tc2 := t_{deckOH} = 1.33\ ft$

$W_{c2} := Conc2Area \cdot Tc2 \cdot \gamma_c = 0\ lbf$

$C2x := 0\ ft$ $\quad$ $C2y := 0\ ft$

$Conc3Area := 596.8739\ ft^2$ *concrete area as measured in cadd*

$Tc3 := t_{deck} = 0.71\ ft$

$W_{c3} := Conc3Area \cdot Tc3 \cdot \gamma_c = 63417.85\ lbf$

$C3x := 40.843\ ft$ $\quad$ $C3y := 3.542\ ft$

$PBU2_{wt} := W_{stl} \cdot 2 + W_{c1} + W_{c2} + W_{c3} = 100040.93\ lbf$

$$PBU2_X := \frac{(Steel1x + Steel2x) \cdot W_{stl} + C1x \cdot W_{c1} + C2x \cdot W_{c2} + C3x \cdot W_{c3}}{PBU2_{wt}} = 40.84\ ft$$

$$PBU2_X := \frac{(Steel1y + Steel2y) \cdot W_{stl} + C1y \cdot W_{c1} + C2y \cdot W_{c2} + C3y \cdot W_{c3}}{PBU2_{wt}} = 3.54\ ft$$

Haunch negligible in centroid calculations

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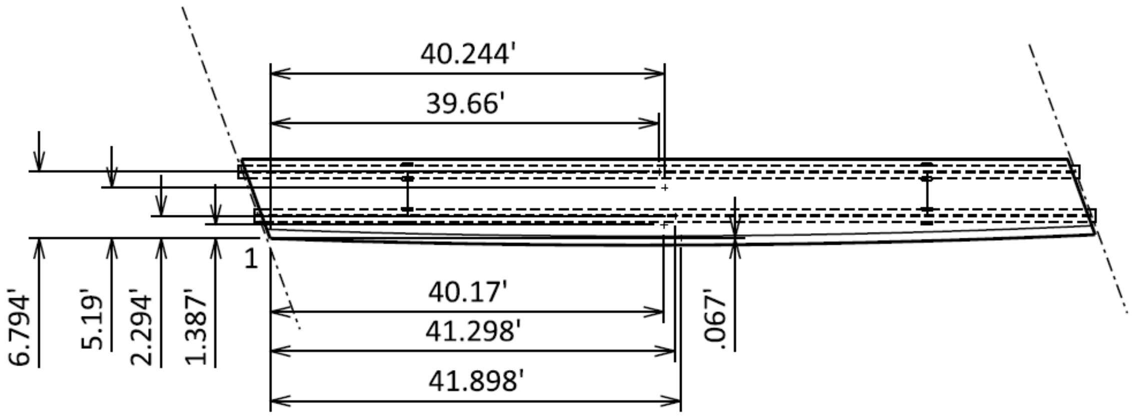
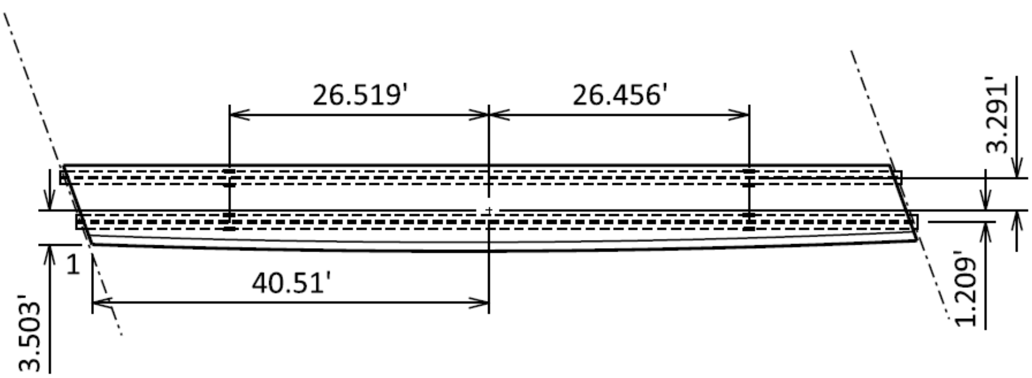
Centroid of PBU #3 (left Fascia) - x & y dimensions are measured from the lower left hand obtuse corner of the precast decking (abutment #1 end, Easterly side)



PBU #3



PBU #3

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$Steel1x := 41.298\ ft$ $Steel1y := 2.294\ ft$

$Steel2x := 39.66\ ft$ $Steel2y := 6.794\ ft$

$Conc1Area := 77.1423\ ft^2$ *concrete area as measured in cadd*

$Tc1 := t_{deckOH} + Curb_{Height} = 1.57\ ft$

$W_{c1} := Conc1Area \cdot Tc1 \cdot \gamma_c = 18200.76\ lbf$

$C1x := 41.898\ ft$ $C1y := 0.067\ ft$

$Conc2Area := 148.9875\ ft^2$ *concrete area as measured in cadd*

$Tc2 := t_{deckOH} = 1.33\ ft$

$W_{c2} := Conc2Area \cdot Tc2 \cdot \gamma_c = 29797.5\ lbf$

$C2x := 40.17\ ft$ $C2y := 1.387\ ft$

$Conc3Area := 488.0373\ ft^2$ *concrete area as measured in cadd*

$Tc3 := t_{deck} = 0.71\ ft$

$W_{c3} := Conc3Area \cdot Tc3 \cdot \gamma_c = 51853.96\ lbf$

$C3x := 40.244\ ft$ $C3y := 5.19\ ft$

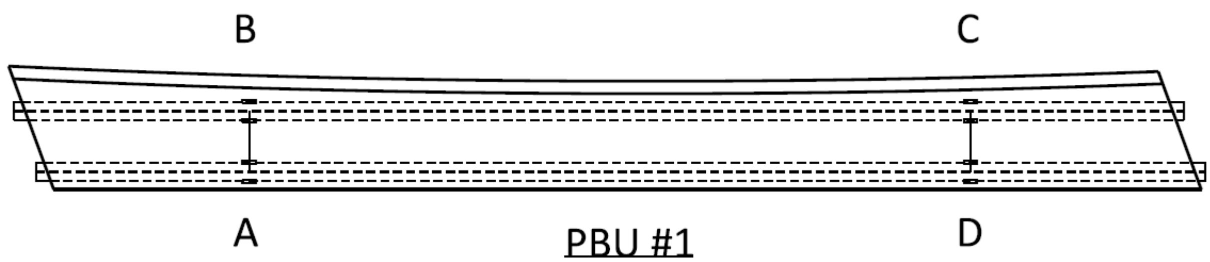
$PBU3_{wt} := W_{stl} \cdot 2 + W_{c1} + W_{c2} + W_{c3} = 136475.31\ lbf$

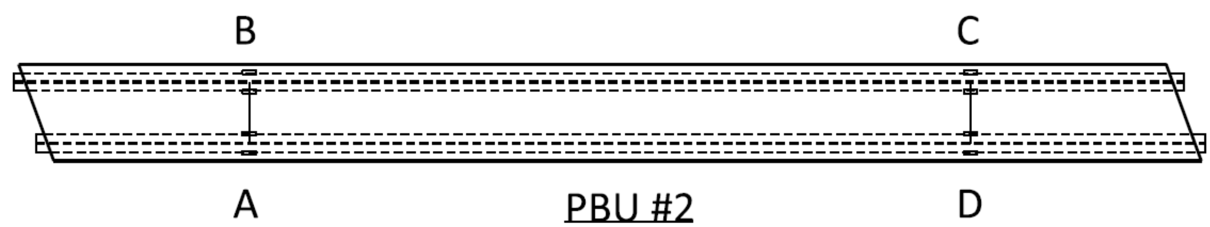
$$PBU3_X := \frac{(Steel1x + Steel2x) \cdot W_{stl} + C1x \cdot W_{c1} + C2x \cdot W_{c2} + C3x \cdot W_{c3}}{PBU3_{wt}} = 40.51\ ft$$

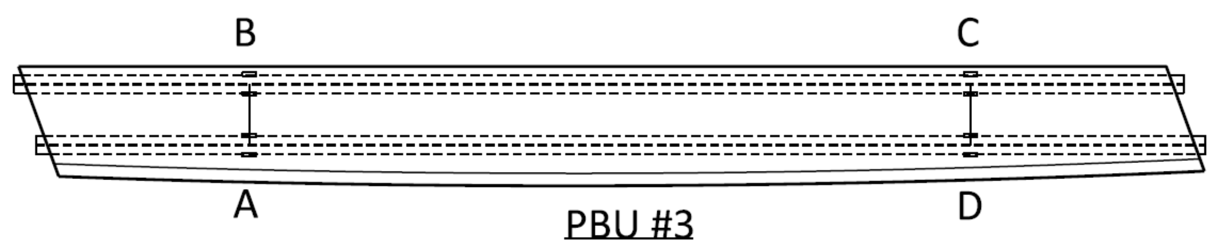
$$PBU3_X := \frac{(Steel1y + Steel2y) \cdot W_{stl} + C1y \cdot W_{c1} + C2y \cdot W_{c2} + C3y \cdot W_{c3}}{PBU3_{wt}} = 3.503\ ft$$

Haunch negligible in centroid calculations

Lifter location ID







$$A_{pbu1} := \frac{PBU1_{wt} \cdot 27.121\ ft}{27.121\ ft + 25.854\ ft} \cdot \frac{1.162\ ft}{4.5\ ft} + \frac{Haunch1}{4} = 18.67\ kip$$

$$B_{pbu1} := \frac{PBU1_{wt} \cdot 27.121\ ft}{27.121\ ft + 25.854\ ft} \cdot \frac{3.338\ ft}{4.5\ ft} + \frac{Haunch1}{4} = 52.14\ kip$$

$$C_{pbu1} := \frac{PBU1_{wt} \cdot 25.854\ ft}{27.121\ ft + 25.854\ ft} \cdot \frac{3.338\ ft}{4.5\ ft} + \frac{Haunch1}{4} = 49.74\ kip$$

$$D_{pbu1} := \frac{PBU1_{wt} \cdot 25.854\ ft}{27.121\ ft + 25.854\ ft} \cdot \frac{1.162\ ft}{4.5\ ft} + \frac{Haunch1}{4} = 17.83\ kip$$

$$B_{pbu1} + C_{pbu1} = 101.88\ kip$$

$$A_{pbu2} := \frac{PBU2_{wt} \cdot 26.487\ ft}{26.487\ ft + 26.487\ ft} \cdot \frac{2.25\ ft}{4.5\ ft} + \frac{Haunch2}{4} = 26.06\ kip$$

$$B_{pbu2} := \frac{PBU2_{wt} \cdot 26.487\ ft}{26.487\ ft + 26.487\ ft} \cdot \frac{2.25\ ft}{4.5\ ft} + \frac{Haunch2}{4} = 26.06\ kip$$

$$C_{pbu2} := \frac{PBU2_{wt} \cdot 26.487\ ft}{26.487\ ft + 26.487\ ft} \cdot \frac{2.25\ ft}{4.5\ ft} + \frac{Haunch2}{4} = 26.06\ kip$$

$$D_{pbu2} := \frac{PBU2_{wt} \cdot 26.487\ ft}{26.487\ ft + 26.487\ ft} \cdot \frac{2.25\ ft}{4.5\ ft} + \frac{Haunch2}{4} = 26.06\ kip$$

$$A_{pbu2} + D_{pbu2} = 52.13\ kip$$

$$A_{pbu3} := \frac{PBU3_{wt} \cdot 26.456\ ft}{26.519\ ft + 26.456\ ft} \cdot \frac{3.291\ ft}{4.5\ ft} + \frac{Haunch3}{4} = 50.64\ kip$$

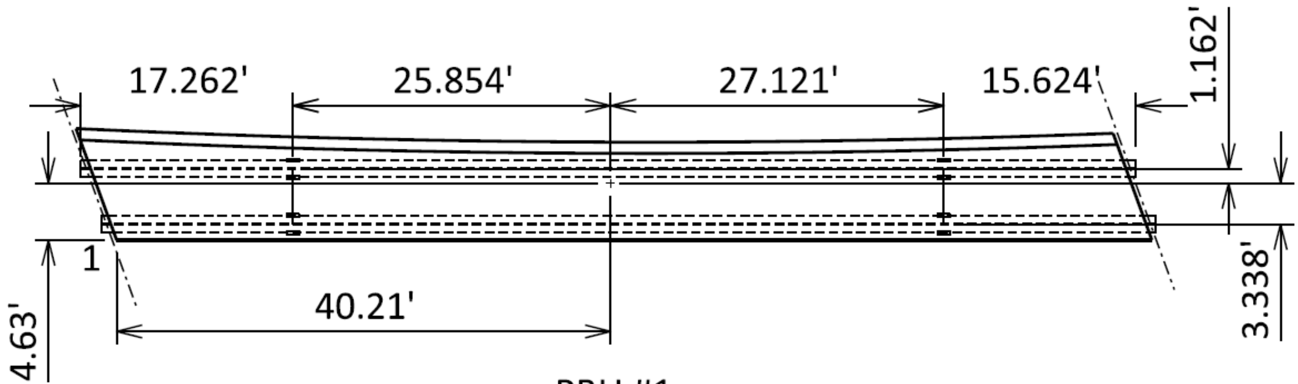
$$B_{pbu3} := \frac{PBU3_{wt} \cdot 26.456\ ft}{26.519\ ft + 26.456\ ft} \cdot \frac{1.209\ ft}{4.5\ ft} + \frac{Haunch3}{4} = 19.1\ kip$$

$$C_{pbu3} := \frac{PBU3_{wt} \cdot 26.519\ ft}{26.519\ ft + 26.456\ ft} \cdot \frac{1.209\ ft}{4.5\ ft} + \frac{Haunch3}{4} = 19.14\ kip$$

$$D_{pbu3} := \frac{PBU3_{wt} \cdot 26.519\ ft}{26.519\ ft + 26.456\ ft} \cdot \frac{3.291\ ft}{4.5\ ft} + \frac{Haunch3}{4} = 50.75\ kip$$

$$A_{pbu3} + D_{pbu3} = 101.39\ kip$$

PBU section governed by PBU #1 by inspection - take load as uniform on the fascia girder use PBU#2 for composite section properties as this is conservative, by adding the heaviest section assuming it is carried by the lightest section



PBU #1

$$w_{eff} := \frac{B_{pbu1} + C_{pbu1}}{17.262\ ft + 25.854\ ft + 27.121\ ft + 15.624\ ft} = 1186.58\ plf$$

$$L_{pbu1} := 17.262\ ft + 25.854\ ft + 27.121\ ft + 15.624\ ft = 85.86\ ft$$

$$R_b := \frac{w_{eff} \cdot L_{pbu1} \cdot \left(\frac{L_{pbu1}}{2} - 15.624\ ft\right)}{25.854\ ft + 27.121\ ft} = 52.52\ kip$$

$$B_{pbu1} = 52.14\ kip$$

$$R_c := \frac{w_{eff} \cdot L_{pbu1} \cdot \left(\frac{L_{pbu1}}{2} - 17.262\ ft\right)}{25.854\ ft + 27.121\ ft} = 49.37\ kip$$

$$C_{pbu1} = 49.74\ kip$$

Distributing the load as a uniform load results in approximately the same reactions, therefor use this for flexural and shear analysis purposes

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Project Notes:
Precast concrete decking cast on fabricated welded plate girders
handling will be done using nylon slings basketing the girders through holes left in concrete decking adjacent to the top flange.
Decking will be cantilevered beyond the end of the truck, impact due to handling and transportation will be taken as 50% (ie 1.5 factor for impact)

Girders are straight - bridge is curved, radius is applied to bridge by varying the overhang, worst case condition will be assumed with the maximum overhang calculated for weight purposes, but the minimum overhange calculated for composite properties, and action

$L_{brg} := 84.974\ ft$ total length of girders bearing to bearing measured along long chord

$skew := 70\ deg$

$$E_{brg} := \frac{5\ in}{\sin(skew)} = 0.44\ ft$$

$L_{oa} := L_{brg} + E_{brg} \cdot 2 = 85.86\ ft$ total length of girders (measured along long chord, 85.00 feet along the 1000 ft radius) + 5 inches to each end of girder from the centerling of bearing to the end of girder

$$OH := \frac{3\ ft + 4\ in - 9\ in}{2} = 1.29\ ft$$ Overhang Used for structural capacity (min)

$K_1 := 1.0$

$\gamma_c := 150\ pcf$

$\gamma_{stl} := 490\ pcf$

$Curb_{Width} := 11\ in$ *Concrete Shear key/ pavement backer width*

$Curb_{Height} := 2.875\ in$ *Concrete Shear key/ pavement backer height*

$f'_c := 4\ ksi$

$$E_c := 33000 \cdot K_1 \cdot \left(\left(\frac{\gamma_c}{\frac{kip}{ft^3}}\right)^{1.5}\right) \cdot \sqrt{f'_c \cdot ksi}$$

$E_c = 3834.25\ ksi$

Girder Properties:

Beam geometry/ Area/ y/ Iox/ Ioy Positive Moment Region

$b_{bf} := 16.0\ \text{in}$

$t_{bf} := 1\ \text{in}$ $A_{bf} := b_{bf} \cdot t_{bf} = 16\ \text{in}^2$ $y_{bf} := \frac{t_{bf}}{2}$ $I_{oxbf} := \frac{b_{bf} \cdot t_{bf}^3}{12}$ $I_{oybf} := \frac{t_{bf} \cdot b_{bf}^3}{12}$

$d_w := 36\ \text{in}$

$t_w := 0.625\ \text{in}$ $A_w := d_w \cdot t_w = 22.5\ \text{in}^2$ $y_w := t_{bf} + \frac{d_w}{2}$ $I_{oxw} := \frac{t_w \cdot d_w^3}{12}$ $I_{oyw} := \frac{d_w \cdot t_w^3}{12}$

$b_{tf} := 16.0\ \text{in}$

$t_{tf} := 1\ \text{in}$ $A_{tf} := b_{tf} \cdot t_{tf} = 16\ \text{in}^2$ $y_{tf} := t_{bf} + d_w + \frac{t_{tf}}{2}$ $I_{oxtf} := \frac{b_{tf} \cdot t_{tf}^3}{12}$ $I_{oytf} := \frac{t_{tf} \cdot b_{tf}^3}{12}$

$d_{beam} := t_{tf} + d_w + t_{bf} = 38\ \text{in}$

$A_g := b_{tf} \cdot t_{tf} + b_{bf} \cdot t_{bf} + d_w \cdot t_w = 54.5\ \text{in}^2$

$y_{bar} := \frac{(A_{bf} \cdot y_{bf} + A_w \cdot y_w + A_{tf} \cdot y_{tf})}{A_g} = 19\ \text{in}$ *Measured from Bottom of Beam to NA*

$I_x := I_{oxbf} + A_{bf} \cdot (y_{bf} - y_{bar})^2 + I_{oxw} + A_w \cdot (y_w - y_{bar})^2 + I_{oxtf} + A_{tf} \cdot (y_{tf} - y_{bar})^2 = 13384.67\ \text{in}^4$

$S_{tf} := \frac{I_x}{(d_{beam} - y_{bar})} = 704.46\ \text{in}^3$ $S_{bf} := \frac{I_x}{y_{bar}} = 704.46\ \text{in}^3$

$I_y := I_{oybf} + I_{oyw} + I_{oytf} = 683.4\ \text{in}^4$

$I_{ttf} := \frac{b_{tf}^3 \cdot t_{tf}}{12} + \frac{(d_w - y_{bar})}{6} \cdot \frac{t_w^3}{12} = 341.39\ \text{in}^4$ *Moment of inertia of the top flagne and 1/6 of the depth of the web in compression about the Y axis*

$A_{ttf} := b_{tf} \cdot t_{tf} + \frac{(d_w - y_{bar})}{6} \cdot t_w = 17.77\ \text{in}^2$ $r_{ttf} := \sqrt{\frac{I_{ttf}}{A_{ttf}}} = 4.38\ \text{in}$ *Radius of Gyration of the top flange and 1/6 of the web depth in compression*

$S := 4.5\ \text{ft}$ *Girder to girder spacing*

Girders - Composite properties:

$E_B := 29000\ ksi$

$E_D := E_c = 3834.25\ ksi$

$n := \frac{E_B}{E_D}$

$A_{deckLT} := \left(\frac{S}{2} + OH\right) \cdot t_{deck} \cdot \frac{E_D}{3 \cdot E_B} = 0.11\ ft^2$

Self weight of deck applied to the long term section (although for our purposes we could likely use the short term section for all of these calculations this is a conservative approach as it will yield higher stresses)

$A_{deckST} := \left(\frac{S}{2} + OH\right) \cdot t_{deck} \cdot \frac{E_D}{E_B} = 0.33\ ft^2$

Impact component applied to the short term section properties

$y_{lt} := \frac{\left(A_{deckLT} \cdot \left(\frac{t_{deck}}{2} + d_{beam}\right) + A_g \cdot y_{bar}\right)}{\left(A_g + A_{deckLT}\right)} = 24.26\ in$

as measured from bottom of bottom flange

$y_{st} := \frac{\left(A_{deckST} \cdot \left(\frac{t_{deck}}{2} + d_{beam}\right) + A_g \cdot y_{bar}\right)}{\left(A_g + A_{deckST}\right)} = 29.86\ in$

as measured from bottom of bottom flange

$d_{tot} := t_{deck} + t_{tf} + d_w + t_{bf} = 46.5\ in$

Haunch is ignored for strength and capacity considerations

$I_{LT} := t_{deck}^{\ 3} \cdot \left(\frac{S}{2} + OH\right) \cdot \frac{E_D}{3 \cdot E_B} + A_{deckLT} \cdot \left(d_{tot} - \frac{t_{deck}}{2} - y_{lt}\right)^2 + I_x + A_g \cdot \left(y_{lt} - y_{bar}\right)^2$

$I_{LT} = 21195.5\ in^4$

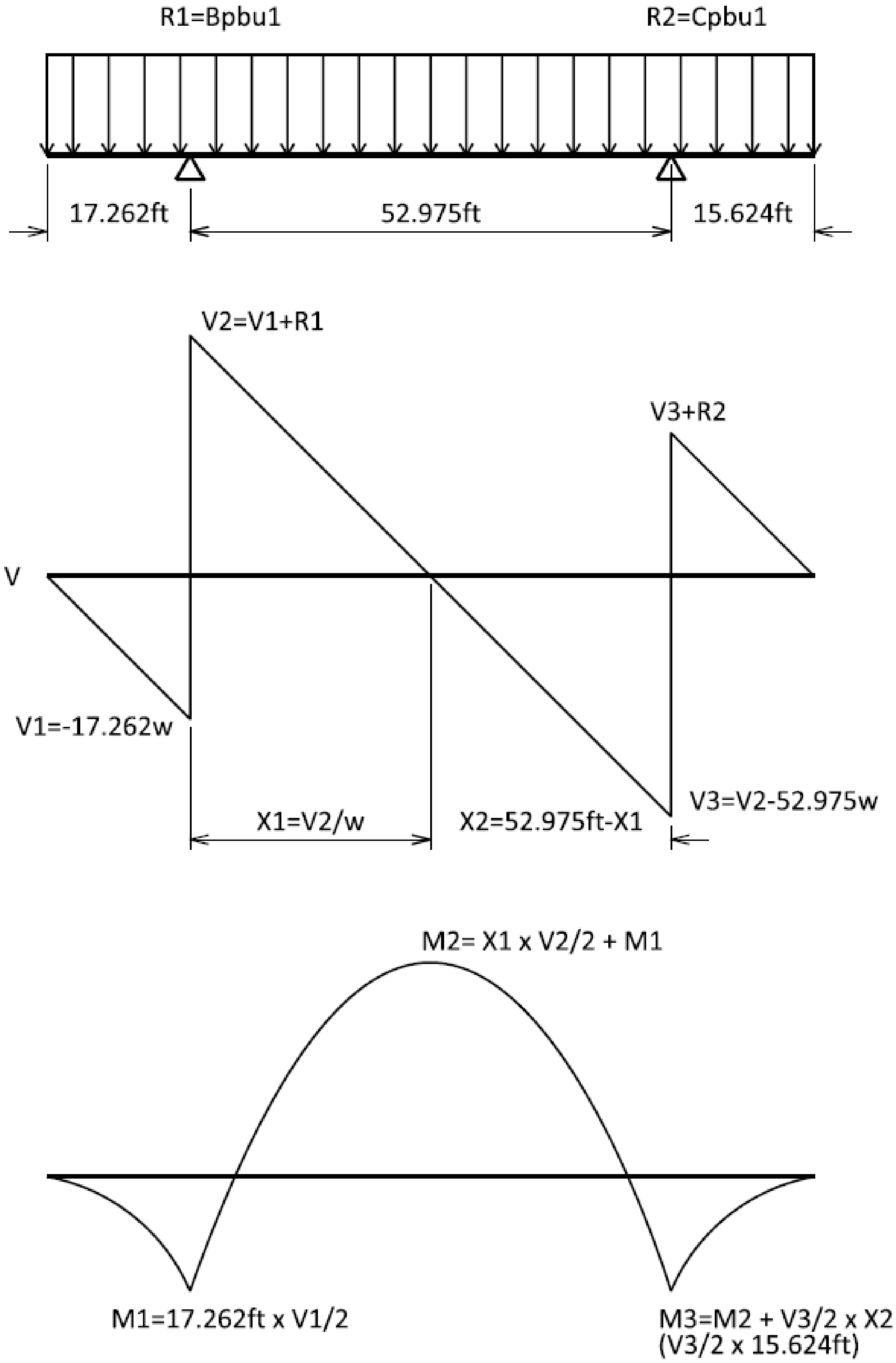
$S_{deckLT} := \frac{I_{LT}}{d_{tot} - y_{lt}} = 952.88\ in^3$ $S_{tfLT} := \frac{I_{LT}}{d_{beam} - y_{lt}} = 1542.21\ in^3$ $S_{bfLT} := \frac{I_{LT}}{y_{lt}} = 873.81\ in^3$

$I_{ST} := t_{deck}^{\ 3} \cdot \left(\frac{S}{2} + OH\right) \cdot \frac{E_D}{E_B} + A_{deckST} \cdot \left(d_{tot} - \frac{t_{deck}}{2} - y_{st}\right)^2 + I_x + A_g \cdot \left(y_{st} - y_{bar}\right)^2$

$I_{ST} = 30595.43\ in^4$

$S_{deckST} := \frac{I_{ST}}{d_{tot} - y_{st}} = 1838.57\ in^3$ $S_{tfST} := \frac{I_{ST}}{d_{beam} - y_{st}} = 3758.26\ in^3$ $S_{bfST} := \frac{I_{ST}}{y_{st}} = 1024.66\ in^3$

Loading Demand Computations:



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Shear Handling

$IM := 50\%$ for transport and handling.

$\gamma_{dc} := 1.5$ for Strength IV configuration (governs by inspection for handling and transit purposes)

$TransitOH1 := 17.262\ ft$ $TransitOH2 := 15.624\ ft$

$L_{support} := L_{oa} - TransitOH1 - TransitOH2 = 52.97\ ft$

$R1 := R_b = 52.52\ kip$ $R2 := R_c = 49.37\ kip$

$V_1 := TransitOH1 \cdot (-w_{eff}) = -20.48\ kip$

$V_2 := R1 + V_1 = 32.03\ kip$

$V_3 := V_2 - L_{support} \cdot w_{eff} = -30.83\ kip$

$V_4 := V_3 + R2 = 18.54\ kip$

$V := \max(\text{abs}(V_1), \text{abs}(V_2), \text{abs}(V_3), \text{abs}(V_4)) = 32.03\ kip$

$M_1 := \frac{V_1}{2} \cdot TransitOH1 = -176.79\ ft \cdot kip$ $X1 := \frac{V_2}{w_{eff}} = 27\ ft$

$M_2 := \frac{V_2}{2} \cdot X1 + M_1 = 255.59\ ft \cdot kip$

$M_{OHstl} := \frac{R1 + R2 - w_{eff} \cdot TransitOH1}{2} \cdot TransitOH1 = 702.55\ ft \cdot kip$

fluid load on steel girder with girder supported at centerline of bearings (as cast in bed) this is a locked in stress in the steel girder resulting in compression in the top flange and tension in the bottom flange) at the transit overhang distance from ends of girders

$M_{midstl} := \frac{w_{eff} \cdot L_{brg}^{\ 2}}{8} = 1070.97\ ft \cdot kip$

fluid load on steel girder with girder supported at centerline of bearings (as cast in bed) this is a locked in stress in the steel girder resulting in compression in the top flange and tension in the bottom flange at midspan)

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$$f_{tfOHLOCKED} := \frac{M_{OHstl}}{S_{tf}} = 11.97 \text{ ksi}$$

$$f_{tfmidLOCKED} := \frac{M_{midstl}}{S_{tf}} = 18.24 \text{ ksi}$$

$$f_{bfOHLOCKED} := \frac{M_{OHstl}}{S_{bf}} = 11.97 \text{ ksi}$$

$$f_{bfmidLOCKED} := \frac{M_{midstl}}{S_{bf}} = 18.24 \text{ ksi}$$

$$f_{stlOHtf} := f_{tfOHLOCKED} \cdot \gamma_{dc} + \frac{M_1 \cdot \gamma_{dc}}{S_{tfLT}} + \frac{M_1 \cdot \gamma_{dc} \cdot IM}{S_{tfST}} = 15.46 \text{ ksi}$$

compression in top flange steel during shipping (factored with impact)

$$f_{stlOHbf} := -f_{tfOHLOCKED} \cdot \gamma_{dc} - \frac{M_1 \cdot \gamma_{dc}}{S_{bfLT}} - \frac{M_1 \cdot \gamma_{dc} \cdot IM}{S_{bfST}} = -12.76 \text{ ksi}$$

tension in bottom flange steel during shipping (factored with impact)

$$f_{deckOH} := \frac{M_1 \cdot \gamma_{dc}}{S_{deckLT} \cdot 3 \cdot n} + \frac{M_1 \cdot \gamma_{dc} \cdot IM}{S_{deckST} \cdot n} = -0.26 \text{ ksi}$$

Factored Strength IV tension in extreme fiber of deck

$$f_{decksvcOH} := \frac{M_1}{S_{deckLT} \cdot 3 \cdot n} + \frac{M_1 \cdot IM}{S_{deckST} \cdot n} = -0.17 \text{ ksi}$$

Service Load tension in extreme fiber of deck at Service I with impact

$$f_{midtf} := f_{tfmidLOCKED} \cdot \gamma_{dc} + \frac{M_2 \cdot \gamma_{dc}}{S_{tfLT}} + \frac{M_2 \cdot \gamma_{dc} \cdot IM}{S_{tfST}} = 30.96 \text{ ksi}$$

compression in top flange steel during shipping (factored with impact) at midspan

$$f_{midbf} := -f_{tfmidLOCKED} \cdot \gamma_{dc} - \frac{M_2 \cdot \gamma_{dc}}{S_{bfLT}} - \frac{M_2 \cdot \gamma_{dc} \cdot IM}{S_{bfST}} = -34.88 \text{ ksi}$$

Tension in bottom flange steel during shipping (factored with impact) at midspan

$$f_{deckmid} := \frac{M_2 \cdot \gamma_{dc}}{S_{deckLT} \cdot 3 \cdot n} + \frac{M_2 \cdot \gamma_{dc} \cdot IM}{S_{deckST} \cdot n} = 0.38 \text{ ksi}$$

Factored Strength IV compression in extreme fiber of deck

$$f_{decksvcmid} := \frac{M_2}{S_{deckLT} \cdot 3 \cdot n} + \frac{M_2 \cdot IM}{S_{deckST} \cdot n} = 0.25 \text{ ksi}$$

Service Load compression in extreme fiber of deck at Service I with impact

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Calculation of yield moment at
Strength IV condition of Girders:

Appendix D6.2.2 Composite Sections in Positive Flexure

$F_y := 50$ ksi

$M_{D1} := M_{midstl} = 1070.97$ ft·kip

$M_{D2} := 0$ ft·kip *for our purposes in determining capacity at shipping d2 is 0*

$$M_{ADtf} := S_{tfST} \cdot \left(F_y - \frac{M_{D1}}{S_{tf}} - \frac{M_{D2}}{S_{tfLT}} \right) = 9945.78 \text{ ft·kip}$$

$$M_{ADbf} := S_{bfST} \cdot \left(F_y - \frac{M_{D1}}{S_{bf}} - \frac{M_{D2}}{S_{bfLT}} \right) = 2711.64 \text{ ft·kip}$$

$$M_{AD} := min \left(M_{ADtf}, M_{ADbf} \right) = 2711.64 \text{ ft·kip}$$

$$M_y := M_{AD} + M_{D1} + M_{D2} = 3782.61 \text{ ft·kip}$$

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Check Plastic Moment for girder:

Appendix D6.1-1 Composite Sections in Positive Flexure

$$P_S := 0.85 \cdot f'_c \cdot \left(\frac{S}{2} + OH\right) \cdot t_{deck} = 1228.25 \text{ kip}$$

$$P_C := F_y \cdot A_{tf} = 800 \text{ kip}$$

per Appendix Pc stands for Compression flange

$$P_W := F_y \cdot A_w = 1125 \text{ kip}$$

$$P_T := F_y \cdot A_{bf} = 800 \text{ kip}$$

per Appendix Pt stands for Tension flange

```
Case := if (P_T + P_W) > (P_C + P_S)     = 2
          ‖ 1
        else if (P_T + P_W + P_C) > P_S
          ‖ 2
        else
          ‖ 3
```

The Plastic Neutral Axis is in the top flange this is not unexpected, the reinforcing steel is neglected as part of the section for the sake of simplicity and conservatism.

$$Y_{pbar} := \frac{t_{tf}}{2} \cdot \left(\frac{(P_W + P_T - P_S)}{P_C} + 1\right) = 0.94 \text{ in}$$

Measured from the top of the top flange

$$d_s := \frac{t_{deck}}{2} + Y_{pbar} = 5.19 \text{ in}$$

$$d_w := t_{tf} - Y_{pbar} + \frac{d_w}{2} = 18.06 \text{ in}$$

$$d_{bf} := t_{tf} - Y_{pbar} + d_w + t_{bf} = 19.13 \text{ in}$$

$$M_p := \frac{P_C}{2 \cdot t_{tf}} \cdot \left(Y_{pbar}^{\ 2} + (t_{tf} - Y_{pbar})^2\right) + (P_S \cdot d_s + P_W \cdot d_w + P_T \cdot d_{bf}) = 3528.88 \text{ ft}\cdot\text{kip}$$

Nominal Flexural resistance per LRFD 6.10.7.1.2

$D_p := Y_{pbar} + t_{deck} = 9.44$ in

$D_{tot} := t_{deck} + d_{beam} = 46.5$ in

$\frac{D_p}{D_{tot}} = 0.2$

Dp/Dtot > 0.1 therefor full plastic moment cannot be reached

$M_n := min\left(M_p \cdot \left(1.07 - 0.7 \cdot \frac{D_p}{D_{tot}}\right), 1.3 \cdot M_y\right) = 3274.67$ ft·kip

Nominal moment capacity per 6.10.7.1.2-3 section during transit section is continuous over interior supports therefore 1.3x yield moment will be the maximum allowable positive moment capacity

$\varphi_f := 1.00$

Resistance Factor from LRFD 6.5.4.2

$M_r := \varphi_f \cdot M_n = 3274.67$ ft·kip

$Sling := 60$ deg

Minimum angle of slings from the horizontal when lifting in the field (induces some positive moment in girders at midspan, has little to no effect on overhang negative moments

$R_{hz} := \frac{\max(R1, R2)}{\tan(Sling)} = 30.32$ kip

$M_{str4} := \gamma_{dc} \cdot M_2 \cdot (100\% + IM) + R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt}) + R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st}) = 690.92$ ft·kip

if $M_r > M_{str4}$
  "Positive Moment Girder Strength IV flexure okay"
else
  "Positive Moment Girder Strength IV flexure no good"

= "Positive Moment Girder Strength IV flexure okay"

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Check Negative moment capacity assuming non composite construction for steel stresses

$R_h := 1.0$ *section is not hybrid yield strength is the same in webs as it is in flanges*

$F_{tf} := \varphi_f \cdot R_h \cdot F_y = 50\ ksi$

flexural capacity of tension flange bottom flange is in tension at all points in the girder regardless of the transit overhang of 16' due to the locked in stresses during the deck placement with the girders supported at CL bearings 6.10.3.2.2

$$f_{tens} := \max\left(abs\left(f_{midbf} + \frac{R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt})}{S_{bfLT}} + \frac{R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st})}{S_{bfST}}\right), abs(f_{stlOHbf})\right) = 33.35\ ksi$$

flexural capacity of compression flange top flange is in compression at all points in the girder regardless of the transit overhang of 16' due to the locked in stresses during the deck placement with the girders supported at CL bearings 6.10.8.2.2

$$\lambda_f := \frac{b_{tf}}{t_{tf} \cdot 2} = 8$$

$\lambda_{pf} := 9.2$

$R_b := 1$

$F_{nc} := R_b \cdot R_h \cdot F_y = 50\ ksi$

$F_{rc} := \varphi_f \cdot F_{nc} = 50\ ksi$

Flexural resistance based on local buckling stress, no need to check lateral torsional buckling as top flange is compression flange and is prevented from twisting by the concrete deck at every location.

$$f_{comp} := \max\left(abs\left(f_{midtf} + \frac{R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt})}{S_{tfLT}} + \frac{R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st})}{S_{tfST}}\right), abs(f_{stlOHtf})\right) = 31.72\ ksi$$

Steel Section is okay during shipping and handling as factored stresses at strength IV with an impact of 50% result in much lower than factored resistance stresses for strength conditions, for both the compression and tension flanges.

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Shear computations: (vertical shear)

Nominal resistance of Unstiffened webs per 6.10.9.2

$D := d_w = 18.06$ in $\quad F_{yw} := F_y = 50$ ksi

$V_p := 0.58 \cdot t_w \cdot D \cdot F_{yw} = 327.42$ kip

$k_u := 5$ for unstiffened panels per 6.10.9.2

$\frac{D}{t_w} = 28.903$ $\quad \varphi_v := 1.0$ Per 6.5.4.2 for Shear

$1.12 \cdot \sqrt{\frac{E_B \cdot k_u}{F_{yw}}} = 60.31 \quad 1.40 \cdot \sqrt{\frac{E_B \cdot k_u}{F_{yw}}} = 75.39$

Because D/tw < 60.31:

$C_u := 1$

$V_{nu} := C_u \cdot V_p = 327.42$ kip

$V_{ru} := \varphi_v \cdot V_{nu} = 327.42$ kip for unstiffened regions

$V_{max} := \gamma_{dc} \cdot (100\% + IM) \cdot \max(\text{abs}(V_1), \text{abs}(V_2), \text{abs}(V_3), \text{abs}(V_4)) = 72.07$ kip *Factored maximum shear with lifters at Transit overhang location*

$R_{strIV} := \gamma_{dc} \cdot (100\% + IM) \cdot \max(R1, R2) = 118.16$ kip *conservatively taking the entire reaction as the shear in the girder*

$V_{ru} := \varphi_v \cdot V_{nu} = 327.42$ kip Maximum factored Shear at ends of girders is < factored shear resistance therefor shear is okay no transverse stiffeners are required.

Check Deck Stresses during lifting
and transport - Strength IV

$$f_c := f_{deckmid} + \frac{R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt})}{S_{deckLT} \cdot 3 \cdot n} + \frac{R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st})}{S_{deckST} \cdot n} = 0.45 \text{ ksi}$$

$\varphi_f := 0.90$ *for flexural compression in tension controlled sections per 5.5.4.2*

$\beta_1 := 0.85$ $f'_c = 4 \text{ ksi}$

$F_c := \varphi_f \cdot f'_c \cdot 0.85 = 3.06 \text{ ksi}$ *>> strength IV compressive stress*

Check Deck Stresses during lifting
and transport - Service I

$$f_c := f_{decksvcmid} + \frac{R_{hz} \cdot (d_{tot} - y_{lt})}{S_{deckLT} \cdot 3 \cdot n} + \frac{R_{hz} \cdot IM \cdot (d_{tot} - y_{st})}{S_{deckST} \cdot n} = 0.3 \text{ ksi}$$

$\varphi_w := 1.0$

$F_{csvc} := 0.60 \cdot \varphi_w \cdot f'_c = 2.4 \text{ ksi}$ *Taken as service limit for fully prestressed concrete at the service limit state for compression resistance at the top of the deck due to shipping and handling 5.9.4.2.1-1*

$F_{t2} := 0.24 \cdot \sqrt{f'_c \cdot \text{ksi}} = 0.48 \text{ ksi}$ *If Deck reinforcement is more than sufficient to resist the total tension force with the mild reinforcement at figured at only 30 ksi stress (0.5Fy)*

$F_{t1} := 0.0948 \cdot \sqrt{f'_c \cdot \text{ksi}} = 0.19 \text{ ksi}$ *allowable temporary tensile stresses in the concrete figured uncracked and without bonded reinforcement (since this is higher than actual tensile stresses anticipated, there is no reason to check the tensile carrying capacity of the longitudinal decking bars)*

$f_t := -f_{decksvcOH} = 0.17 \text{ ksi}$ *Actual concrete decking tension stress figured on the uncracked section*

Lifting with any transit overhang length less than that calculated here will result in a smaller stress than calculated above, to ensure completion we will check the stresses with lifing only at the ends of the PBU although the intention during PBU erection is to lift with a similar basketed detail around the girders but at a point 3' from the ends of the girders, stresses will clearly be enveloped by those previously calculated and these

$$M_{ustrIV} := \frac{(100\% + IM) \cdot \gamma_{dc} \cdot L_{brg}^{2} \cdot w_{eff}}{8} + R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt}) + R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st}) = 2525.53 \ ft \cdot kip$$

$$M_r = 3274.67 \ ft \cdot kip$$

$$f_{deckstrIV} := \frac{\gamma_{dc} \cdot L_{brg}^{2} \cdot w_{eff}}{8 \cdot S_{deckLT} \cdot 3\ n} + \frac{IM \cdot \gamma_{dc} \cdot L_{brg}^{2} \cdot w_{eff}}{8 \cdot S_{deckST} \cdot n} + \frac{R_{hz} \cdot \gamma_{dc} \cdot (d_{tot} - y_{lt})}{S_{deckLT} \cdot 3 \cdot n} + \frac{R_{hz} \cdot \gamma_{dc} \cdot IM \cdot (d_{tot} - y_{st})}{S_{deckST} \cdot n} = 1.66 \ ksi$$

$$F_c = 3.06 \ ksi$$

Note compressive stresses on the decking are conservatively calculated assuming the deck is acting to support it's own self weight, this is obviously not the case as the entire fluid load is carried on the steel girders so actual compressive stresses due to handling will actually be less than this. (please note that the entire fluid load has been applied to the steel non composite section in calculating the compressive stresses in the top flange in previous calculations)

$$f_{decksvc} := \frac{L_{brg}^{2} \cdot w_{eff}}{8 \cdot S_{deckLT} \cdot 3\ n} + \frac{IM \cdot L_{brg}^{2} \cdot w_{eff}}{8 \cdot S_{deckST} \cdot n} + \frac{R_{hz} \cdot (d_{tot} - y_{lt})}{S_{deckLT} \cdot 3 \cdot n} + \frac{R_{hz} \cdot IM \cdot (d_{tot} - y_{st})}{S_{deckST} \cdot n} = 1.11 \ ksi$$

$$F_{csvc} = 2.4 \ ksi$$

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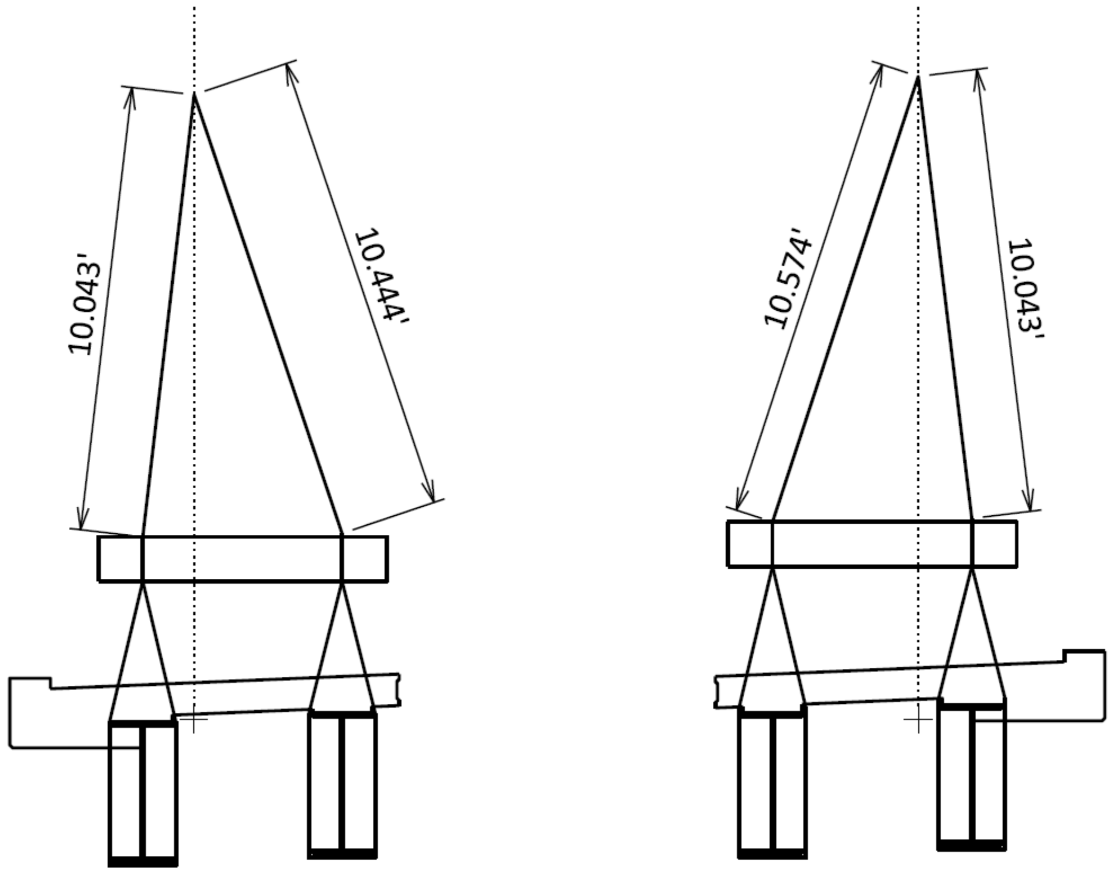
$\text{MaxSling} := \sqrt{\max(R1, R2)^2 + R_{hz}^2} = 60.64\ \text{kip}$

Use 35T shackles min



Slings may be equal leg for the PBU #2, for PBU #1 and #3 in order to ensure the pieces fly close to level, ensure that the sling closest to the centerline of the bridge is about 5.5" longer than the sling closest to the fascia, this can be done by adding an extra shackle. Shackles and rigging for the fascia girders to be 35T minimum

$B_{pbu3} = 19.1\ \text{kip}$

$C_{pbu3} = 19.14\ \text{kip}$

$\text{MinSling} := \sqrt{B_{pbu2}^2 + \left(\frac{B_{pbu2}}{\tan(60\ \text{deg})}\right)^2} = 30.1\ \text{kip}$

All other slings and shackles may be 25T slings