Final Design Calculations for
Cast in Place Deck on Steel Girders – VE Alternate

In the Town of

Rockingham, Vt.

BRF-0126(12)

Prepared for:

Cold River Bridges

By:

Calderwood Engineering etc

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Cover photo of Cavendish
Courtesy of Cold River Bridges
BRF-0146(13)

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Project: Rockingham VT Bridge
Contractor: Cold River Bridges

Value Engineering Design: Calderwood Engineering
Design Computations by: Eric T. Calderwood, PE
Design Check Computations by: Gregory MacAlister, EIT

Project Notes:

VTrans Structural Design Manual

Calculate fatigue distribution factors similarly but with only one lane loaded.
5.1.1.1.1: Concrete, high performance class A for bridge decks placed on steel
5.1.2.6: Minimum Concrete Cover
5.2.1.1.1: Cast in Place Concrete Minimum deck thickness, paved deck, 8.5"
5.2.5: Minimum Reinforcement, #5 bars at 12" for reinforcing steel

Railing may be a TL-2 railing based on traffic volume and other parameters per Vtrans standard, however we will use the TL-4 loads for Vertical concrete parapet to check the overhang reinforcing

$f'_c := 4\ ksi$ $\quad t_{deck} := 9\ in$ $\quad IM := 33\%$ *Dynamic Load Allowance 3.6.2.1-1*

$f_y := 60\ ksi$ $\quad A_5 := 0.31\ in^2$ *Area of a single #5 bar*

$\varepsilon_{cmax} := 0.003\ \frac{in}{in}$ $\quad A_6 := 0.44\ in^2$ *Area of a single #6 bar*

$\varepsilon_{smintc} := -0.005\ \frac{in}{in}$ $\quad S_{ctoc} := 10\ ft + 6\ in = 10.5\ ft$

$C_{topbars} := 2.5\ in$ $\quad b_{minf} := 16\ in$ *Minimum flange width of supporting girders*

$C_{btmbars} := 1.5\ in$ $\quad OH := 2\ ft + 7\ in = 2.5833\ ft$

$B_w := 10\ in$ *Barrier or curb width* $\quad t_{pvt} := 3.25\ in$ *thickness of pavement (& membrane)*

$\omega_{railing} := B_w \cdot (2\ ft + t_{pvt}) \cdot 150\ pcf + 25\ plf = 0.3089\ klf$

$m_p := 1.20$ *Multi presence factor for one lane loaded per 3.6.1.1.2* $\quad \varphi_{tc} := 0.90$ *Tension controlled resistance factor for reinforced concrete sections 5.5.4.2.1*

$E_c := 1820 \cdot \sqrt{f'_c \cdot ksi} = 3640\ ksi$ *C5.4.2.4*

$E_s := 29000\ ksi$ *5.4.3.2* $\quad t_w := \frac{7}{16}\ in$

$n := \frac{E_s}{E_c} = 7.967$

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$S := S_{ctoc} = 10.5\ ft$

$h := t_{deck} = 9\ in$

$\gamma_{ll} := 1.75$ $\gamma_{llmin} := 0.00$

$\gamma_{dc} := 1.25$ $\gamma_{dcmin} := 0.90$

$\gamma_{dw} := 1.50$ $\gamma_{dwmin} := 0.65$

$\gamma_{svc} := 1.00$

Critical section for Negative moments 4.6.2.1.6 25% of the flange width from the centerline of girder.

$25\% \cdot b_{minf} = 4\ in$

From Appendix A4 in the AASHTO LRFD Bridge Design Specifications: (includes Dynamic Load & multi presence factors, live load only)

$P_{mll} := 7.17\ \frac{ft \cdot kip}{ft}$ *Max + moment*

$P3in_{nll} := -7.64\ \frac{ft \cdot kip}{ft}$

$P6in_{nll} := -6.77\ \frac{ft \cdot kip}{ft}$

$P_{nll} := P3in_{nll} + \frac{\langle P3in_{nll} - P6in_{nll} \rangle}{3\ in - 6\ in} \cdot (4\ in - 3\ in) = -7.35\ \frac{ft \cdot kip}{ft}$

Negative Design moment, interpolated between 3 in & 6 in for design section at 4 in from cl web

$P_{nlloh} := -1\ \frac{kip}{ft} \cdot \langle OH - B_w - 12\ in \rangle \cdot (100\% + IM) \cdot m_p$

$P_{nlloh} = -1.197\ \frac{ft \cdot kip}{ft}$

3.6.1.3.4 Deck overhang load where continuous barrier is in place and overhang is less than 6 ft outside wheel loads may be replaced wiht a 1klf load placed 1 ft from the barrier.

$P_{mll} = 7.17\ \frac{ft \cdot kip}{ft}$

$P_{nll} = -7.35\ \frac{ft \cdot kip}{ft}$

$P_{nlloh} = -1.197\ \frac{ft \cdot kip}{ft}$

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For uniformly loaded condition: (1k/lf/ftwidth):

$$l := S = 10.5\ ft$$

$$\omega := 1\ \frac{klf}{ft}$$

$$M_{upos} := \frac{\omega \cdot l^2}{12} = 9.1875\ \frac{ft \cdot kip}{ft}$$

$$M_{uneg} := \frac{-\omega \cdot l^2}{10} = -11.025\ \frac{ft \cdot kip}{ft}$$

$$M_{uoh} := \frac{-\omega \cdot OH^2}{2} = -3.3368\ \frac{ft \cdot kip}{ft}$$

DC & DW :

$$\omega_{dc} := t_{deck} \cdot 150\ pcf = 0.1125\ \frac{klf}{ft}$$

$$\omega_{dw} := t_{pvt} \cdot 145\ pcf = 0.0393\ \frac{klf}{ft}$$

$$M_{dcp} := \frac{\omega_{dc}}{\omega} \cdot M_{upos} = 1.0336\ \frac{ft \cdot kip}{ft}$$

$$M_{dwp} := \frac{\omega_{dw}}{\omega} \cdot M_{upos} = 0.3608\ \frac{ft \cdot kip}{ft}$$

$$M_{dcn} := \frac{\omega_{dc}}{\omega} \cdot M_{uneg} = -1.2403\ \frac{ft \cdot kip}{ft}$$

$$M_{dwn} := \frac{\omega_{dw}}{\omega} \cdot M_{uneg} = -0.433\ \frac{ft \cdot kip}{ft}$$

$$M_{dcnoh} := \frac{\omega_{dc}}{\omega} \cdot M_{uoh} = -0.3754\ \frac{ft \cdot kip}{ft}$$

$$M_{dwnoh} := \frac{\omega_{dw}}{\omega} \cdot M_{uoh} = -0.131\ \frac{ft \cdot kip}{ft}$$

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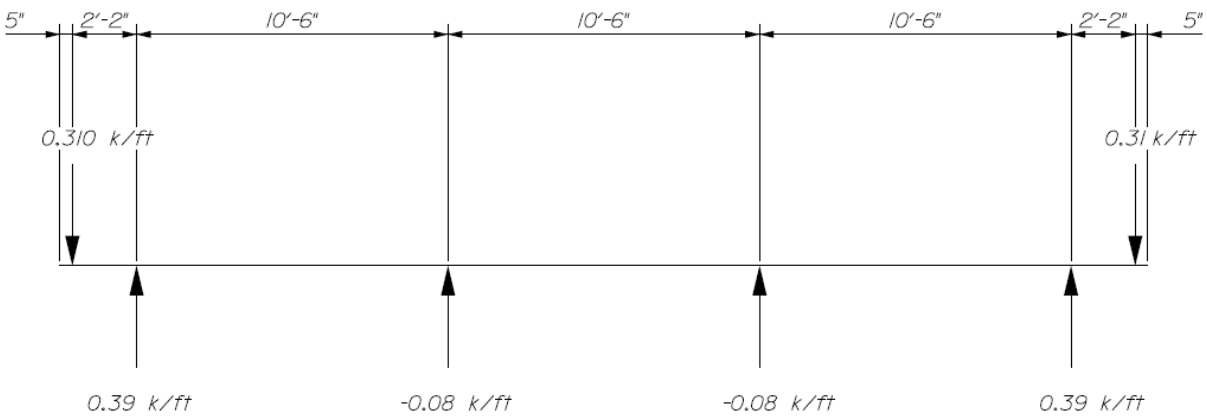
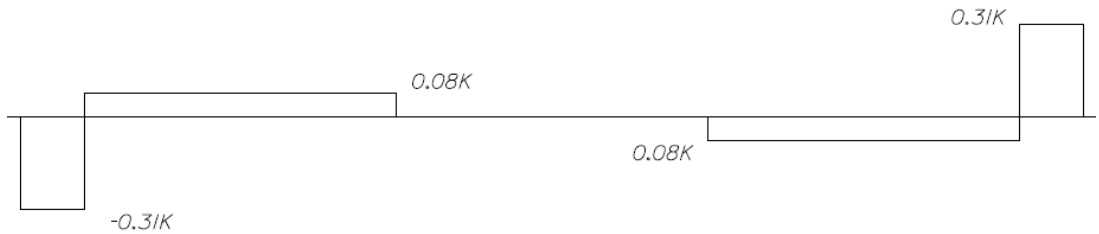


REACTIONS DUE TO RAILING LOAD



SHEAR DIAGRAM DUE TO RAILING LOAD

$$M_{railnoh} := -\omega_{railing} \cdot \left(OH - \frac{B_w}{2}\right) = -0.6692 \ \frac{ft \cdot kip}{ft}$$

$$M_{railn} := 0.08 \ klf \cdot \left(S - \frac{b_{minf}}{4}\right) + M_{railnoh} = 0.1441 \ \frac{ft \cdot kip}{ft}$$

$$M_{railp} := 0.08 \ klf \cdot S + M_{railnoh} = 0.1708 \ \frac{ft \cdot kip}{ft}$$

Maximum Positive & Negative moments at strength I:

$$M_{str1pos} := \gamma_{ll} \cdot P_{mll} + \gamma_{dc} \cdot \left(M_{railp} + M_{dcp}\right) + \gamma_{dw} \cdot M_{dwp} = 14.5942 \ \frac{ft \cdot kip}{ft}$$

$$M_{str1neg} := \gamma_{ll} \cdot P_{nll} + \gamma_{dcmin} \cdot M_{railn} + \gamma_{dc} \cdot M_{dcn} + \gamma_{dw} \cdot M_{dwn} = -14.9326 \ \frac{ft \cdot kip}{ft}$$

$$M_{str1oh} := \gamma_{ll} \cdot P_{nlloh} + \gamma_{dc} \cdot \left(M_{railnoh} + M_{dcnoh}\right) + \gamma_{dw} \cdot M_{dwnoh} = -3.597 \ \frac{ft \cdot kip}{ft}$$

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$A_{stt} := 0.62 \ \frac{in^2}{ft}$ *Area of steel transverse top*

$A_{stb} := 0.62 \ \frac{in^2}{ft}$ *Area of steel transverse bottom*

$A_{ps} := 0 \ \frac{in^2}{ft}$ $f_{pu} := 0 \ ksi$ $f_s := f_y = 60 \ ksi$

$A'_s := 0 \ \frac{in^2}{ft}$ $f'_s := 0 \ ksi$ $f'_c = 4 \ ksi$

$\varphi_{topbars} := \frac{5}{8} \ in$ $\varphi_{btmbars} := \frac{5}{8} \ in$

$d_{stop} := t_{deck} - C_{topbars} - \frac{\varphi_{topbars}}{2} = 6.1875 \ in$

$d_{sbtm} := t_{deck} - C_{btmbars} - \frac{\varphi_{btmbars}}{2} = 7.1875 \ in$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c < 4 \ ksi \\ 0.85 - (f'_c - 4 \ ksi) \cdot 0.05 \cdot \frac{1}{ksi} & \text{else} \end{cases} = 0.85$$

$b := 12 \ \frac{in}{ft}$ $k := 0.28$

Flexural Capacity of
positive moment region:

Assume fs=fy, rectangular distribution, and neglect compression reinforcing

$d_p := 1 \ in$ *dp set to 1in although Aps=0 in^2 as there is no prestressing steel*

$$c_{btm} := \frac{A_{ps} \cdot f_{pu} + A_{stb} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 1.073 \ in$$

$\varepsilon_{cmax} = 0.003 \ \frac{in}{in}$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{btm}}\right) \cdot (c_{btm} - d_{sbtm}) = -0.0171 \ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_{npos} := A_{stt} \cdot f_s \cdot \left(d_{sbtm} - \frac{\beta_1 \cdot c_{btm}}{2}\right) = 20.868 \ \frac{ft \cdot kip}{ft}$ *Positive moment nominal capacity*

$M_{rpos} := \varphi_{tc} \cdot M_{npos} = 18.7812 \ \frac{ft \cdot kip}{ft}$

$M_{str1pos} = 14.5942 \ \frac{ft \cdot kip}{ft}$

Moment Capacity is > Str I positive moment therefor okay for str. I Positive Moment

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Flexural Capacity of
negative moment region:

Assume fs=fy, rectangular distribution, and neglect compression reinforcing

$$c_{top} := \frac{A_{ps} \cdot f_{pu} + A_{stt} \cdot f_s - A'_s \cdot f'_s}{\left(0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}\right)} = 1.073 \text{ in}$$

$$\varepsilon_{cmax} = 0.003 \frac{\text{in}}{\text{in}}$$

$$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{top}}\right) \cdot \left(c_{top} - d_{stop}\right) = -0.0143 \frac{\text{in}}{\text{in}}$$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$$M_{nneg} := -A_{stt} \cdot f_s \cdot \left(d_{stop} - \frac{\beta_1 \cdot c_{top}}{2}\right) = -17.768 \frac{\text{ft}\cdot\text{kip}}{\text{ft}}$$

Negative moment nominal capacity

$$M_{rneg} := \varphi_{tc} \cdot M_{nneg} = -15.9912 \frac{\text{ft}\cdot\text{kip}}{\text{ft}}$$

Moment Capacity is > Str I negative moment therefor okay for str. I Negative moment & Str I overhang moment

$$M_{str1neg} = -14.9326 \frac{\text{ft}\cdot\text{kip}}{\text{ft}}$$

$$M_{str1oh} = -3.597 \frac{\text{ft}\cdot\text{kip}}{\text{ft}}$$

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Check Minimum Reinforcing
requirements: per section 5.7.3.3.2

$$S_c := \frac{\left(t_{deck}^{\ 2} \cdot 12\ \frac{in}{ft}\right)}{6} = 162\ \frac{in^3}{ft}$$

Section modulus of 12inch strip of deck (gross section uncracked)

$\gamma_1 := 1.6$ *for other than precast segmental structures*

$\gamma_2 := 1.0$ *for unbonded tendons (no prestressing force applied gamma 2x fcpe = 0*

$\gamma_3 := 0.67$ *for A615 grade 60 reinforcing per 5.7.3.3.2*

$f_{cpe} := 0\ ksi$ *there is no prestressing force*

$S_{nc} := S_c$ *non composite section is the same as the composite section*

$f_r := 0.24 \cdot \sqrt{f'_c \cdot ksi} = 0.48\ ksi$ *modulus of rupture per 5.4.2.6*

$M_{dnc} := 0\ \frac{ft \cdot kip}{ft}$ *all load is composite (ie there is no non composite section)*

$$M_{cr} := \gamma_3 \cdot \left(\left(\gamma_1 \cdot f_r + \gamma_2 \cdot f_{cpe}\right) \cdot S_c - M_{dnc} \cdot \left(\frac{S_c}{S_{nc}} - 1\right)\right) = 6.9466\ \frac{ft \cdot kip}{ft}$$

$$M_{rpos} = 18.7812\ \frac{ft \cdot kip}{ft}$$

$$M_{rneg} = -15.9912\ \frac{ft \cdot kip}{ft}$$

Resisting moment positive & Negative is > Mcr therefor minimum reinforcement is adequate per section 5.7.3.3.2

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Check Crack Control
Reinforcement per section 5.7.3.4

Maximum Positive & Negative moment service loads:

$$M_{svcpos} := \gamma_{svc} \cdot P_{mll} + \gamma_{svc} \cdot (M_{railp} + M_{dcp}) + \gamma_{svc} \cdot M_{dwp} = 8.7352 \ \frac{ft \cdot kip}{ft}$$

$$M_{svcneg} := \gamma_{svc} \cdot P_{nll} + \gamma_{svc} \cdot M_{railn} + \gamma_{svc} \cdot M_{dcn} + \gamma_{svc} \cdot M_{dwn} = -8.8791 \ \frac{ft \cdot kip}{ft}$$

$$M_{svcoh} := \gamma_{svc} \cdot P_{nlloh} + \gamma_{svc} \cdot (M_{railnoh} + M_{dcnoh}) + \gamma_{svc} \cdot M_{dwnoh} = -2.3726 \ \frac{ft \cdot kip}{ft}$$

Cracked Neutral Axis Positive Flexure:

$$A_{sbtrans} := A_{stb} \cdot n = 4.9396 \ \frac{in^2}{ft}$$ *area of steel transformed to concrete*

$$c_{crpos} := 2.0555 \ in$$ *distance to cracked neutral axis (assumed)*

$$y_{sb} := C_{btmbars} + \frac{\varphi_{btmbars}}{2} = 1.8125 \ in$$

$$y_{concpos} := t_{deck} - \frac{c_{crpos}}{2} = 7.9723 \ in$$

$$A_{concpos} := c_{crpos} \cdot 12 \ \frac{in}{ft} = 24.666 \ \frac{in^2}{ft}$$

$$y_{barpos} := \frac{A_{concpos} \cdot y_{concpos} + A_{sbtrans} \cdot y_{sb}}{A_{concpos} + A_{sbtrans}} = 6.9445 \ in$$

$$c_{crpos} := t_{deck} - y_{barpos} = 2.0555 \ in$$

$$I_{crpos} := \frac{\left(c_{crpos}^{\ 3} \cdot 12 \ \frac{in}{ft}\right)}{12} + A_{concpos} \cdot (y_{concpos} - y_{barpos})^2 + A_{sbtrans} \cdot (y_{sb} - y_{barpos})^2 = 164.8336 \ \frac{in^4}{ft}$$

$$S_{crposstl} := \frac{I_{crpos}}{(y_{barpos} - y_{sb}) \cdot n} = 4.0314 \ \frac{in^3}{ft} \qquad S_{crposconc} := \frac{I_{crpos}}{t_{deck} - y_{barpos}} = 80.1924 \ \frac{in^3}{ft}$$

$$f_{sspos} := \frac{M_{svcpos}}{S_{crposstl}} = 26.0012 \ ksi$$

$$f_{concpos} := \frac{M_{svcpos}}{S_{crposconc}} = 1.3071 \ ksi$$

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$\gamma_e := 1.0$ *Exposure factor for Class I Exposure condition*

$d_{cpos} := y_{sb} = 1.8125$ in

$h := t_{deck} = 9$ in

$\beta_{sb} := 1 + \frac{d_{cpos}}{0.7 \cdot \langle h - d_{cpos} \rangle} = 1.3602$ *5.7.3.4*

$s_{maxpos} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_{sb} \cdot f_{sspos}} - 2 \cdot d_{cpos} = 16.1668$ in *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition. 6 inches per bar << 16.1668 in therefor okay*

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Cracked Neutral Axis Negative Flexure:

$A_{sttrans} := A_{stt} \cdot n = 4.9396 \ \frac{in^2}{ft}$ *area of steel transformed to concrete*

$c_{crneg} := 1.8826 \ in$ *distance to cracked neutral axis (assumed)*

$y_{st} := C_{topbars} + \frac{\varphi_{topbars}}{2} = 2.8125 \ in$

$y_{concneg} := t_{deck} - \frac{c_{crneg}}{2} = 8.0587 \ in$

$A_{concneg} := c_{crneg} \cdot 12 \ \frac{in}{ft} = 22.5912 \ \frac{in^2}{ft}$

$$y_{barneg} := \frac{A_{concneg} \cdot y_{concneg} + A_{sttrans} \cdot y_{st}}{A_{concneg} + A_{sttrans}} = 7.1174 \ in$$

$c_{crneg} := t_{deck} - y_{barneg} = 1.8826 \ in$

$$I_{crneg} := \frac{\left(c_{crneg}^{\ 3} \cdot 12 \ \frac{in}{ft}\right)}{12} + A_{concneg} \cdot \left(y_{concneg} - y_{barneg}\right)^2 + A_{sttrans} \cdot \left(y_{st} - y_{barneg}\right)^2 = 118.2296 \ \frac{in^4}{ft}$$

$S_{crnegstl} := \frac{I_{crneg}}{\left(y_{barneg} - y_{st}\right) \cdot n} = 3.4472 \ \frac{in^3}{ft}$ $\quad S_{crnegconc} := \frac{I_{crneg}}{t_{deck} - y_{barneg}} = 62.8022 \ \frac{in^3}{ft}$

$f_{ssneg} := \frac{-M_{svcneg}}{S_{crnegstl}} = 30.9092 \ ksi$

$f_{concneg} := \frac{-M_{svcneg}}{S_{crnegconc}} = 1.6966 \ ksi$

$d_{cneg} := y_{st} = 2.8125 \ in$

$h := t_{deck} = 9 \ in$

$\beta_{st} := 1 + \frac{d_{cneg}}{0.7 \cdot \left(h - d_{cneg}\right)} = 1.6494$ *5.7.3.4*

$s_{maxneg} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_{st} \cdot f_{ssneg}} - 2 \cdot d_{cneg} = 8.1058 \ in$ *5.7.3.4 for maximum reinforcing steel bar spacing based on class I exposure condition 6 in spacing << 8.1058 in spacing therefor okay*

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Longitudinal steel:

$S_{ftofclr} := S - b_{minf} = 9.1667\ ft$

$S_{eff} := S_{ftofclr} + \frac{b_{minf} - t_w}{2} = 9.8151\ ft$ *9.7.2.3 Effective span Length*

for primary steel perpendicular to traffic: 9.7.3.2

$\frac{220\%\ ft}{\sqrt{S_{eff} \cdot ft}} = 70.2223\%$ *>67% therefor use 67%*

$A_{sdistbtm} := 67\% \cdot A_{stb} = 0.4154\ \frac{in^2}{ft}$

$A_5 = 0.31\ in^2$

$S_{max} := \frac{A_5 \cdot 12\ \frac{in}{ft}}{A_{sdistbtm}} = 8.9552\ in$ *space bars at 8 in centers*

$S_{lbtm} := 8\ in$

$A_{lbtm} := \frac{A_5 \cdot 12\ \frac{in}{ft}}{S_{lbtm}} = 0.465\ \frac{in^2}{ft}$

$A_{ltop} := 0.31\ \frac{in^2}{ft}$ *space top longitudinal bars at 12in centers per Vtrans standard minimum reinforcement*

Note: for negative moment reinforcing steel required over interior supports reference steel calculations. Due to tensile stresses in concrete deck at svc II conditions over the piers being less than 90% modulus of rupture therefor 1% of the deck area requirement does not apply and the typical distribution reinforcement may continue through the negative moment region

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Check Temp & Shrinkage Steel:

$b_{oa} := 36\ ft + 8\ in = 440\ in$ $\quad h = 9\ in$ $\quad f_y = 60\ ksi$

$$A_{st\&s} := \frac{1.30\ \frac{kip}{in \cdot ft} \cdot b_{oa} \cdot h}{2 \cdot (b_{oa} + h) \cdot f_y} = 0.09555\ \frac{in^2}{ft}$$ *5.10.8-1*

$A_{st\&s} < 0.11\ \frac{in^2}{ft}$ *therefor*

$A_{st\&s} := 0.11\ \frac{in^2}{ft}$

5.10.8-2 area of steel provided = 0.62 square inches per foot each face transverse 0.31 square inches per foot minimum longitudinal top face and 0.465 square inches per foot bottom face therefor temp & shrinkage okay

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$A_{stlbarhor} := 3 \cdot 0.2\ in^2 = 0.6\ in^2$

$y_{bbar} := 2\ in + 0.5\ in + \frac{0.5\ in}{2} = 2.75\ in$ $\qquad b_{barv} := 24\ in + t_{pvt} = 27.25\ in$

$d_{sbar} := B_w - y_{bbar} = 7.25\ in$

$c_{bar} := \frac{A_{stlbarhor} \cdot f_s}{(0.85 \cdot f'_c \cdot \beta_1 \cdot b_{barv})} = 0.457\ in$

$\varepsilon_{cmax} = 0.003\ \frac{in}{in}$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{bar}}\right) \cdot (c_{bar} - d_{sbar}) = -0.0446\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_w := A_{stlbarhor} \cdot f_s \cdot \left(d_{sbar} - \frac{\beta_1 \cdot c_{bar}}{2}\right) = 21.1672\ ft{\cdot}kip$ *Barrier wall moment capacity*

$Z_{yhss4x3} := 3.12\ in^3$ $\qquad Z_{yhss2x2} := 0.584\ in^3$

$Z_{rail} := Z_{yhss4x3} + Z_{yhss2x2} = 3.704\ in^3$ $\qquad F_{yrail} := 50\ ksi$

$M_b := Z_{rail} \cdot F_{yrail} = 15.4333\ ft{\cdot}kip$ *Plastic moment capacity of top railings. (assuming the highest grade tube steel results in a higher plastic moment and a higher demand on the deck design, therefor is conservative for purposes of deck design)*

$A_{stlbarver} := \frac{0.2\ in^2 \cdot 12\ \frac{in}{ft}}{8\ in} = 0.3\ \frac{in^2}{ft}$ *#4 bars at 8 inch spacing*

$y_{bbarv} := 3\ in + \frac{0.5\ in}{2} = 3.25\ in$ $\qquad b = 12\ \frac{in}{ft}$ $\qquad \varepsilon_{cmax} = 0.003\ \frac{in}{in}$

$d_{sbarv} := B_w - y_{bbarv} = 6.75\ in$

$c_{barv} := \frac{A_{stlbarver} \cdot f_s}{(0.85 \cdot f'_c \cdot \beta_1 \cdot b)} = 0.519\ in$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{barv}}\right) \cdot (c_{barv} - d_{sbarv}) = -0.036\ \frac{in}{in}$ *tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy*

$M_c := A_{stlbarver} \cdot f_s \cdot \left(d_{sbarv} - \frac{\beta_1 \cdot c_{barv}}{2}\right) = 9.7941\ \frac{ft{\cdot}kip}{ft}$ *Barrier wall horizontal cantilevered moment capacity per linear foot*

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$L_t := 3.5\ ft$ *From Table A13.2-1 for TL-4 Railing* $H_w := b_{barv} = 27.25\ in$

$$L_{ci} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H_w \cdot (M_b + M_w)}{M_c}} = 10.1732\ ft$$ *A13.3.1-2 interior wall segments*

$$R_{wi} := \frac{2}{2 \cdot L_{ci} - L_t} \cdot \left(8 \cdot (M_b + M_w) + \frac{M_c \cdot L_{ci}^2}{H_w}\right) = 87.7546\ kip$$ *A13.3.1-1 at interior wall segments*

$$L_{ce} := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H_w \cdot \frac{M_b + M_w}{M_c}} = 5.1483\ ft$$ *A13.3.1-4 at end panels or joints*

$$R_{we} := \frac{2}{2 \cdot L_{ce} - L_t} \cdot \left(M_b + M_w + \frac{M_c \cdot L_{ce}^2}{H_w}\right) = 44.4095\ kip$$ *A13.3.1-3 at end panels or joints*

$$R_w := \max(R_{we}, R_{wi}) = 87.7546\ kip$$

$$L_c := L_{ci} = 10.1732\ ft$$

$$T_{deck} := \frac{R_w}{L_c + 2 \cdot H_w} = 5.9636\ \frac{kip}{ft} \qquad M_c = 9.7941\ \frac{ft \cdot kip}{ft}$$

$$M_{dcbarfob} := \omega_{railing} \cdot \frac{B_w}{2} = 0.1287\ \frac{ft \cdot kip}{ft}$$

$$M_{deckfob} := t_{deck} \cdot 150\ pcf \cdot \frac{B_w^2}{2} = 0.0391\ \frac{ft \cdot kip}{ft}$$

$$M_{fob} := -1 \cdot \left(M_c + T_{deck} \cdot \frac{t_{deck}}{2} + 1.25 \cdot (M_{dcbarfob} + M_{deckfob})\right) = -12.2402\ \frac{ft \cdot kip}{ft}$$ *Collision force & moment at face of barrier*

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$A_{stt} = 0.62 \ \frac{in^2}{ft}$ *Area of steel in top straight bars transverse per foot with #5 bars at 6" spacing*

$A_{htt} := 0.62 \ \frac{in^2}{ft}$ *Additional hooked bar #5 at 6" on center at overhang*

Check Development length of #5 straight bars:

$$l_{db} := \max\left(\frac{1.25}{in} \cdot A_5 \cdot f_y, 0.4 \cdot \frac{in^2}{kip} \cdot \varphi_{topbars} \cdot f_y\right) = 15 \ in$$

5.11.2.1.1 Tension development length

Modification of development lengths:

$m_{ep} := 1.2$ *Epoxy Coated bars with adequate cover and spacing (3d cvr & 6in spacing) 5.11.2.1.2*

$l_d := m_{ep} \cdot l_{db} = 18 \ in$

without hooks adjust the Area of Reinforcing steel to accomodate incomplete develolopment development length available = 10 in - 2 in = 8in

$l_{soht} := 8 \ in$

$A_{sohtadj} := \frac{l_{soht}}{l_d} \cdot A_{stt} = 0.2756 \ \frac{in^2}{ft}$ *Adjusted Area of steel to account for incomplete development*

$A_{Tdeduct} := \frac{T_{deck}}{60 \ ksi} = 0.0994 \ \frac{in^2}{ft}$ *Reduction of area of steel required to carry collision tension force Tdeck*

Check Development length of #5 hooked bars:

$$l_{hb} := \frac{38 \cdot ksi \cdot \varphi_{topbars}}{\sqrt{f'_c \cdot ksi}} = 11.875 \ in$$

$l_h := l_{hb} \cdot m_{ep} = 14.25 \ in$

$l_{hoht} := 8 \ in$

$A_{hohtadj} := \frac{l_{hoht}}{l_h} \cdot A_{htt} = 0.3481 \ \frac{in^2}{ft}$

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$A_{fob} := A_{sohtadj} - A_{Tdeduct} + A_{hohtadj} = 0.5242 \ \frac{in^2}{ft}$

$d_{stop} = 6.1875 \ in$ $\quad b = 12 \ \frac{in}{ft}$ $\quad \varepsilon_{cmax} = 0.003 \ \frac{in}{in}$

$c_{soh} := \frac{A_{fob} \cdot f_s}{(0.85 \cdot f'_c \cdot \beta_1 \cdot b)} = 0.907 \ in$

$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{soh}}\right) \cdot (c_{soh} - d_{stop}) = -0.0175 \ \frac{in}{in}$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$M_{nfob} := A_{fob} \cdot f_s \cdot \left(d_{stop} - \frac{\beta_1 \cdot c_{soh}}{2}\right) = 15.2081 \ \frac{ft \cdot kip}{ft}$

$\varphi_{ee} := 1.0$ *Extreme event resistance factor = 1.0*

$M_{rfob} := \varphi_{ee} \cdot M_{nfob} = 15.2081 \ \frac{ft \cdot kip}{ft}$

deck flexural resistance capacity at face of barrier with partially developed straight #5 bars and partially developed #5 hooked bars both at 6" spacing > M at face of barrier therefor okay

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Check Moment due to overhang and due to collision force at centerline of exterior girder:

$$M_{str1oh} = -3.597 \ \frac{ft \cdot kip}{ft}$$

$$M_{oh} := M_{str1oh} - T_{deck} \cdot \frac{t_{deck}}{2} - M_c = -15.6275 \ \frac{ft \cdot kip}{ft}$$

$$A_{soh2} := A_{stt} + A_{htt} = 1.24 \ \frac{in^2}{ft}$$

$$c_{soh2} := \frac{A_{soh2} \cdot f_s}{(0.85 \cdot f'_c \cdot \beta_1 \cdot b)} = 2.145 \ in$$

$$\varepsilon_s := \left(\frac{\varepsilon_{cmax}}{c_{soh2}}\right) \cdot (c_{soh2} - d_{stop}) = -0.0057 \ \frac{in}{in}$$

tensile strain is > 0.005 therefor behavior is tensile controlled, and fs=fy

$$M_{noh} := A_{soh2} \cdot f_s \cdot \left(d_{stop} - \frac{\beta_1 \cdot c_{soh2}}{2}\right) = 32.7096 \ \frac{ft \cdot kip}{ft}$$

$$\varphi_{ee} := 1.0$$

Extreme event resistance factor = 1.0

$$M_{roh} := \varphi_{ee} \cdot M_{noh} = 32.7096 \ \frac{ft \cdot kip}{ft}$$

deck flexural resistance capacity at face of barrier with fully developed straight #5 bars and fully developed #5 hooked bars both at 6" spacing > M at centerline of girder therefor okay

Use standard 180 degree hooks and ensure they project the development length beyond the center of the exterior girder

Lap Splice lengths: As provided is not twice As required and assume 100 percent of Area of steel is spliced at location (ie class C splice) 5.11.5.3.1

$$l_d = 18 \ in$$

$$l_{lap\#5} := 1.7 \cdot l_d = 30.6 \ in$$

Specify 36" lap splice length for all lap splices for #5 bars in the deck