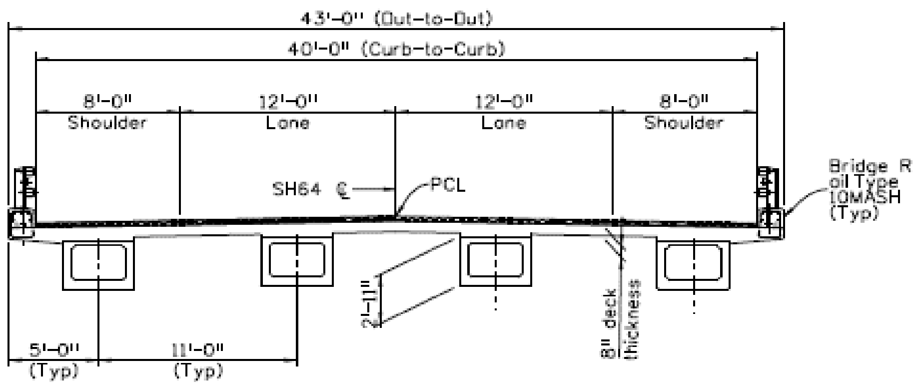
# APPENDIX A
# EXAMPLE 6 - DECK DESIGN INCLUDING TYPE 10 MASH RAIL COLLISION
## EXAMPLE 6.1 - DECK DESIGN

### GENERAL INFORMATION

Based on AASHTO LRFD Bridge Design Specifications 9.6.1, there are 3 methods of deck analysis:

1. Approximate Elastic Method, or "Equivalent Strip" Method (AASHTO 4.6.2.1)
2. Refined Methods (AASHTO 4.6.3.2)
3. Empirical Design Method (AASHTO 9.7.2)

This design example uses the Approximate Elastic Method (Equivalent Strip Method), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.




TYPICAL SECTION

### MATERIAL AND SECTION PROPERTIES

| | | | | |
|---|---|---|---|---|
| Structure Type | | CIP Concrete Deck | | |
| Girder Spacing, maximum | $S_{Gdr}$= | **11.0** | ft | |
| Number of girders | $N_{Gdr}$= | **4** | ea | |
| Overall Deck width | $W_{deck}$= | **43.0** | ft | |
| Deck slab thickness | $t_{deck}$= | **8** | in | |
| Overhang thickness (average) | $t_{OH}$= | **9.67** | in | |
| Concrete top cover | $c_{Top}$= | **2.0** | in | AASHTO T 5.10.1-1 & BDM 5.4.3 |
| Concrete bottom cover | $c_{Bot}$= | **1.0** | in | AASHTO T 5.10.1-1 |
| Wearing surface | $t_{WS}$= | **3.0** | in | |
| | | | | |
| Concrete strength | $f'_c$= | **4.5** | ksi (Concrete Class D or G compressive strength) | |
| Reinforcement strength | $f_y$= | **60.0** | ksi (Minimum yield strength of grade 60 steel) | |
| Concrete density | $W_c$= | **0.150** | kcf | |
| Deck overlay density | $W_{WS}$= | **0.147** | kcf | BDM 3.4.2 |
| Allowance for future utilities | $W_{util}$= | **0.005** | ksf | BDM 3.4.3 |
| Resistance factors | $\varphi_{STR}$ = | **0.9** | (strength limit state) | |
| | $\varphi_{EE}$ = | **1.0** | (extreme event limit state) | |
| Correction factor for source aggregate | $K_1$= | **1** | | |
| Modulus of elasticity of reinforcement | $E_s$= | **29000.0** | ksi | AASHTO 5.4.3.2 |
| Modulus of elasticity of concrete | $E_c$= | **4435.3** | ksi | AASHTO 5.4.2.4 |
| $E_c = 120{,}000 K_1 W_c^2 f'^{0.33}_c$ | | | | |
| Modular ratio | $n=E_s/E_c$= | **6.54** | | |
| Girder Type | | Box Girder | | |
| Girder web thickness | web= | **4.0** | in | |
| Girder top flange width | flange= | **48.0** | in | |

| | | | |
|---|---|---|---|
| Barrier Type | | **Type 10MASH** | |
| CY of concrete for barrier section | $A_B$= | **0.06** | CY/ft |
| Barrier Weight | $W_{barrier}$= | **0.461** | kip/ft |

(Refer to CDOT bridge Worksheet B-606-10MASH for more details)

## UNFACTORED DEAD LOADS

Based on Table 3-22c, Continuous Beams Moment and Shear Coefficients - Equal Spans, Equally Loaded, in terms of wl2, +M =0.080 and -M = 0.100 and will be used for this design

+Moment in terms of $wl^2$ **0.08**
-Moment in terms of $wl^2$ **0.10**

$W_{deck}$= 8.00 in /12 * 0.15 kcf = 0.1 klf
$W_{WS}$= 3.00 in /12 * 0.147 kcf = 0.037 klf

Positive Moment
$+M_{deck}$= 0.100 klf * (11.00 ft)^2 * 0.08 = 0.968 k-ft/ft
$+M_{WS}$= 0.037 klf * (11.00 ft)^2 * 0.08 = 0.355 k-ft/ft

Negative Moment
$-M_{deck}$= 0.100 klf * (11.00 ft)^2 * 0.10 = 1.21 k-ft/ft
$-M_{WS}$= 0.037 klf * (11.00 ft)^2 * 0.10 = 0.444 k-ft/ft

## UNFACTORED LIVE LOADS

Live load moment can be determined by using AASHTO LRFD Bridge Design Specifications Appendix A4 T.A4-1. This table lists positive and negative moments per unit width of the deck with various girder spacings and various distances from the design section to the centerline of girders. This table is based on the equivalent strip method and interpolation is allowed when needed.

Deck superstructure type **b** AASHTO T4.6.2.2.1-1
Design section = At the face of the supporting component 24.00 in AASHTO 4.6.2.1.6

Girder spacing, S= 11.0 ft
Maximum Live Loads per unit width:
Positive Moment from LL $+M_{LL}$= **7.46** kip-ft/ft AASHTO T. A4-1
Negative Moment from LL $-M_{LL}$= **4.52** kip-ft/ft AASHTO T. A4-1

## FACTORED DESIGN LOADS

Concrete decks must be investigated for strength, service and extreme limit states. Fatigue and fracture limit states do not need to be investigated (AASHTO 9.5).

$M_u = \eta\ [\gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + m\ \gamma_{LL}(M_{LL}+ IM)]$
$\eta$ = 1.0 load modifier
$\gamma$ - load factors specified in AASHTO T.3.4.1-1, T.3.4.1-2
m - multiple presence factor, included in values from AASHTO T. A4-1
IM - dynamic load allowance, included in values from AASHTO T. A4-1

| | Load Factors | | | Design Moments | |
|---|---|---|---|---|---|
| Load Combination | $\gamma_{DC\_max}$ | $\gamma_{DW\_max}$ | $\gamma_{LL}$ | $+M_{LL}$ | $-M_{LL}$ |
| Strength I | 1.25 | 1.5 | 1.75 | 14.80 | -10.09 |
| Service I | 1 | 1 | 1 | 8.78 | -6.17 |

Note - it is conservative to use minimum load factors for positive values of $M_{100}$ and $M_{200}$ and negative values of $M_{150}$.

Controlling positive factored moment +Mu = 14.80 kip-ft/ft
Controlling negative factored moment -Mu = -10.09 kip-ft/ft

## DECK SLAB STRENGTH DESIGN

Design of deck reinforcement, including flexural resistance, limits of reinforcement, and control of cracking is based on AASHTO LRFD Bridge Design Specifications 5.7.3 (typical rectangular beam design). The following design method can be used for normal weight concrete with specified compressive strengths up to 15.0 ksi. Refer to Section 9, Deck and Deck Systems, of this BDM for information about acceptable deck reinforcement sizes and spacing.

Width of the design section $b =$ **12.0** in.

Resistance factor for tension-controlled section $\varphi_{STR} =$ 0.9 AASHTO 5.5.4.2

*Positive Moment Capacity (bottom reinforcement)*

| | | | | |
|---|---|---|---|---|
| Try | Bar size | # | **5** | |
| | Bar spacing | $s =$ | **6.0** | in. |
| | Bar diameter | $d_b =$ | 0.625 | in. |
| | Bar area | $A_b =$ | 0.31 | in.$^2$ |

Area of steel per design strip

$A_S = b\,(A_b / s) =$ 12.0 in. * 0.310 in.$^2$/ 6.0 in. = 0.62 in.$^2$

Effective depth of section

$d_S = t_{Deck} - c_{Bot} - 1/2\ d_b =$ 8.0 in. - 1.0 in. - 0.625 in. / 2 = 6.69 in.

Depth of equivalent stress block

$$a = \frac{A_S f_y}{0.85 f'_c b} =$$ 0.62 in.$^2$ * 29000.0 ksi / (0.85 * 1.0 ksi * 12 in.) = 0.81 in.

Factored flexural resistance $$+\varphi M_n = \varphi A_S f_y \left(d_S - \frac{a}{2}\right) =$$

= 0.90 * 0.62 in.$^2$* 60.0 ksi * (6.69 in. - 0.81 in. / 2) / 12 in./ft. = 17.53 kip-ft.

$Check + \varphi M_n > +M_u$: 17.53 > 14.80 **OK**

*Negative Moment Capacity (top reinforcement)*

| | | | | |
|---|---|---|---|---|
| Try | Bar size | # | **5** | |
| | Bar spacing | $s =$ | **5.0** | in. |
| | Bar Diameter | $d_b =$ | 0.625 | in. |
| | Bar Area | $A_b =$ | 0.31 | in.$^2$ |

Area of steel per 1.00 ft. design strip

$A_S = B\,(A_b / s) =$ 12 in. * 0.310 in.$^2$/ 5.00 in. = 0.74 in.$^2$

Effective depth of section

$d_S = t_{Deck} - c_{Top} - 1/2\ d_b =$ 8.0 in. - 2.0 in. - 0.625 in. / 2 = 5.69 in.

Depth of equivalent stress block

$$a = \frac{A_s f_y}{0.85 f'_c b} =$$ 0.74 in$^2$* 60.0 ksi / (0.85 * 4.5 ksi * 12 in.) = 0.97 in.

Factored flexural resistance $$-\varphi M_n = \varphi A_s f_y \left(d_s - \frac{a}{2}\right) =$$

= 0.90 * 0.74in.$^2$* 60.0 ksi * (5.69 in. - 0.97 in. / 2) / 12 in./ft. = 17.41 kip-ft.

$Check\ -\varphi M_n > -M_u$: 17.41 > 10.09 **OK**

*Minimum Reinforcement* AASHTO 5.6.3.3

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, Mr = φMn, at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment
- Cracking moment

Cracking moment $$M_{cr} = \gamma_3\left[(\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$ AASHTO 5.6.3.3-1

When simplified by removing all values applicable to prestressed and noncomposite sections, this equation becomes the following: $M_{cr} = \gamma_3 \gamma_1 f_r S_c$

Where: AASHTO 5.6.3.3

| | | | |
|---|---|---|---|
| Flexural cracking variability factor | $\gamma_1$ = | **1.6** | (non-segmental brg.) |
| Ratio of specified min. yield strength to ultimate tensile strength | $\gamma_3$ = | **0.67** | (A615 steel) |
| Concrete density modification factor | $\lambda$ = | **1.0** | AASHTO 5.4.2.8 |

Modulus of rupture $f_r = 0.24\lambda\sqrt{f_c'} =$ 0.509 ksi AASHTO 5.4.2.6

Section modulus of design section $S_c = \dfrac{bh^2}{6} = \dfrac{bt_{Deck}^2}{6} =$ 12.0in. * (8.0 in.)$^2$ / 6 = 128 in.$^3$

Check Positive Moment reinforcement

$Check + \varphi M_n \geq min$

1.33 (+$M_u$) = 1.33 * 14.80 kip-ft. = 19.68 kip-ft.

$M_{cr}$ = 0.67 * 1.60 * 0.51 ksi * 128.0 in.$^3$/ 12 in./ft. = 5.82 kip-ft.

17.53 > 5.82 **OK**

Check Negative Moment reinforcement

$Check - \varphi M_n \geq min$

1.33 (-$M_u$) = 1.33 * 10.09 kip-ft. = 13.42 kip-ft.

$M_{cr}$ = 0.67 * 1.60 * 0.51 ksi * 128.0 in.$^3$/ 12 in./ft. = 5.82 kip-ft.

17.41 > 5.82 **OK**

## CONTROL OF CRACKING AT SERVICE LIMIT STATE

Cracking is controlled by the spacing of mild steel reinforcement in the layer closest to the tension face, which shall satisfy the following (need not be less than 5.00 in.):

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$ AASHTO 5.6.7-1

In which: $\gamma_e$ = **1.00** - exposure factor (1.0 for Class 1 and 0.75 for Class 2) (assume waterproofing membrane is used)

$b_s$ - ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

$f_{ss}$ - calculated tensile stress in mild steel reinforcement at the service limit state ( ≤ 0.60 $f_y$ ksi)

$d_c$ - thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto. For calculation purposes, $d_c$ need not be taken greater than 2 in. plus the bar radius

Check Cracking at the Bottom of Deck (spacing of Positive Moment reinforcement):

$d_c = c_{Bot} + 1/2\ d_b =$ 1.00 in. + 0.625 in. / 2 = 1.31 in.

$\beta_S = 1 + \dfrac{d_C}{0.7(t_{Deck} - d_C)} =$ 1 + 1.31 in. / [0.7 (8.0 in. - 1.31 in.)] = 1.28

Tension reinforcement ratio $\rho = \dfrac{A_S}{bd_S} =$ 0.62 in.$^2$ / (12 in. * 6.69 in.) = 0.008

$k = \sqrt{2n\rho + (n\rho)^2} - n\rho =$ 0.271

$j = 1 - k/3 =$ 0.910

$f_{ss} = \dfrac{+M_{u\_service}}{A_S j d_S} =$ 8.78 kip-ft. * 12in./ft. / (0.62 in.$^2$ * 0.91 * 6.69 in.) = 27.95 ksi

$s_{max} = \dfrac{700\gamma_e}{\beta_S f_{SS}} - 2d_C =$ 700 * 1.00 / (1.28 * 27.95 ksi) - 2 * 1.31in. = 16.94 in.

Spacing of positive moment reinforcement used in the design = 6.00 in.

$Check\ spacing\ used \leq s_{max}$: 6.00 < 16.94 **OK**

Check Cracking at Top of Deck (spacing of Negative Moment reinforcement):

$d_c = c_{Top} + 1/2\ d_b =$ 2.0 in. + 0.625 in. / 2 = 2.31 in.

$\beta_s = 1 + \frac{d_C}{0.7(t_{Deck} - d_C)} =$ 1+ 2.31 in. / [0.7 * (8.0 in. - 2.31 in.)] = 1.58

Tension reinforcement ratio $\rho = \frac{A_S}{bd_S} =$ 0.74 in.$^2$ / (12 in. * 5.69 in.) = 0.011

Modular ratio $n = E_S / E_C =$ 29000 ksi / 4435 ksi = 6.54

$k = \sqrt{2n\rho + (n\rho)^2} - n\rho =$ 0.313

$j = 1 - k/3 =$ 0.896

$f_{ss} = \frac{-M_{u\_service}}{A_S j d_S} =$ 6.17 kip-ft. * 12in./ft. / (0.74 in.$^2$ * 0.90 * 5.69 in.) = 19.55 ksi

$s_{max} = \frac{700\gamma_e}{\beta_S f_{SS}} - 2d_C =$ 700 * 1.00 / (1.58 * 19.55 ksi) - 2 * 2.31 in. = 18.03 in.

Spacing of negative moment reinforcement used in the design = 5.00 in.

$Check\ spacing\ used \leq s_{max}$: 5.00 < 18.03 **OK**

## LONGITUDINAL REINFORCEMENT

Minimum reinforcement is required in all directions to accommodate shrinkage and temperature changes near the surface of the slab. Longitudinal reinforcement on each face shall meet the following:

$$A_S \geq \frac{1.3b\ t_{Deck}}{2(b + t_{Deck})f_y}$$ AASHTO 5.10.6-1

$$0.11 \leq A_S \leq 0.60$$ AASHTO 5.10.6-2

$A_{s,min} =$ 1.3 * 12.0 in.* 8.0 in. / [2 (12.0 in. + 8.0 in.) 60.0 ksi] = 0.052 in.$^2$/ft.

$A_{s,min} =$ 0.11 in.$^2$/ft. - controls

Per Section 9.6 of the CDOT BDM, the minimum longitudinal reinforcing steel in the top of the concrete bridge deck shall be #4 @ 6.00 in. Longitudinal reinforcement in the bottom of the deck slab can be distributed as a percentage of the primary reinforcement for positive moment.

Top reinforcement try **# 4 @ 6.00 in** on center: $A_S =$ 0.40 in.$^2$/ft.

$Check\ A_S \geq A_{S\ min}$ **OK**

Effective span length $S = S_{Gdr} - girder\ width$ 11.0 ft. - 48.0in. / 12in./ft. = 7 ft.

AASHTO 9.7.2.3

Amount of reinforcement required in secondary direction in the bottom of the slab

$\frac{220}{\sqrt{S}} \leq 67\%$ $\frac{220}{\sqrt{S}} =$ 83% Use - 67% AASHTO 9.7.3.2

Area of primary reinforcement for positive moment = 0.62 in.$^2$/ft.

Required area of bottom longitudinal steel: $A_{S\_Req} =$ 67% * 0.62 in.$^2$/ft.= 0.42 in.$^2$/ft.

Bottom reinforcement try **# 5 @ 8.00 in.** on center: $A_S =$ 0.465 in.$^2$/ft.

$Check\ A_S \geq A_{S\ min}$ **OK**

$Check\ A_S \geq A_{S\_Req}$ **OK**

| **DECK SECTION SUMMARY** | | | |
|---|---|---|---|
| Deck thickness | | | **8.00 in.** |
| Top Transverse Reinforcement | **# 5** | **@** | **5.00 in.** |
| Bottom Transverse Reinforcement | **# 5** | **@** | **6.00 in.** |
| Top Longitudinal Reinforcement | **# 4** | **@** | **6.00 in.** |
| Bottom Longitudinal Reinforcement | **# 5** | **@** | **8.00 in.** |

## EXAMPLE 6.2 - TYPE 10 MASH STRENGTH DESIGN

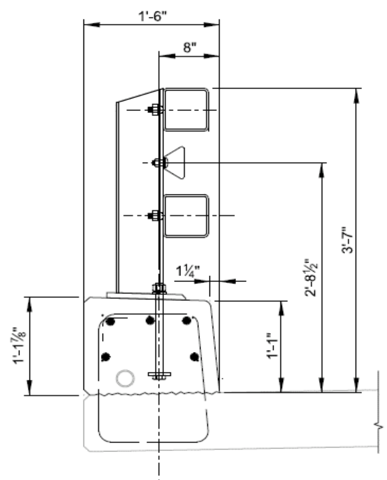
### GENERAL INFORMATION

CDOT Bridge Rail Type 10MASH consists of a concrete parapet and a metal rail. The resistance to transverse vehicular impact loads shall be determined as specified in AASHTO LRFD Bridge Design Specifications A13.3.3. End impact is not considered. See CDOT Worksheet B-606-10MASH for barrier details.

| | | | | |
|---|---|---|---|---|
| Overall barrier height | $H_B$ = | **43.0** | in. | |
| Concrete cover | c = | **2.0** | in. | |
| Resistance factors | $\varphi_{EE}$ = | **1** | (Extreme Event) | AASHTO 1.3.2.1 |
| | $\varphi_S$ = | **0.8** | (A325 bolts in shear) | AASHTO 6.5.4.2 |
| | $\varphi_T$ = | **0.8** | (A325 bolts in tension) | AASHTO 6.5.4.2 |
| Test level | | **TL-4** | | AASHTO T.A13.2-1 |
| Transverse design force | $F_t$ = | **80.0** | kips | See table below |
| Impact force distribution | $L_t$ = | **5.0** | ft. | See table below |

### CONCRETE PARAPET

| | | | |
|---|---|---|---|
| Height | $H_W$ = | **13.4375** | in. |
| Width at base | d = | **18.0** | in. |
| Concrete Compressive Strength | f'c = | **4.5** | ksi |
| Reinforcing Steel | fy = | **75.0** | ksi |



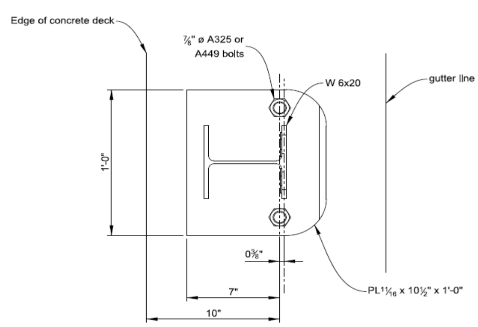

### RAIL POST

| | | | | |
|---|---|---|---|---|
| Type | | **W6x20** | | |
| Steel grade | | **ASTM A-572, Grade 50** | | |
| Post spacing | L = | **10** | ft. (max) | |
| Effective height | $H_R$ = | **32.5** | in. | |
| Area of post | $A_{Post}$ = | **5.87** | $in.^2$ | |
| Web depth | D = | **5.47** | in. | |
| Web thickness | $t_W$ = | **0.26** | in. | |
| Flange thickness | $t_F$ = | **0.37** | in. | |
| Flange width | $b_f$= | **6.02** | | |
| Depth of W beam | $d_b$= | **6.2** | | |
| | Fy (post) = | **50** | ksi | AISC Table 1-1 |
| | Zx-x (post) = | **14.9** | $in.^3$ | |
| $M_n=M_p=F_yZ$ (F7-1 AISC Manual) | $M_{post}$= | **62.08** | kip-ft | |

### RAIL TUBES

AISC Table 1-12

| | | | |
|---|---|---|---|
| Type | | **HSS 6x6x1/4** | |
| Steel grade | | **ASTM A-1085** | |
| Area of one tube | $A_{Tube}$ = | **5.59** | $in.^2$ |
| Number of tubes | $n_{Tubes}$ = | **2** | ea. |
| | Fy (tube) = | **50.0** | ksi |
| | Z (tube) = | **11.2** | $in.^3$ |
| $M_n=M_p=F_yZ$ (F7-1 AISC Manual) | $M_p$= | **93.33** | kip-ft |




### BASE PLATE

| | | | |
|---|---|---|---|
| Width of base plate | $W_b$ = | **12.0** | in. |
| Thickness of base plate | $t_b$ = | **0.6875** | in. |
| Distance to bolts | $d_{bo}$ = | **10.0** | in. |
| Bolt diameter | Ø = | **0.875** | in. |
| Min tensile strength | $F_{ub}$ = | **120.0** | ksi |
| Number of bolts | $n_b$ = | **2** | |

**Design Forces for Traffic Railings**

| Test Level | Rail Height (in.) | $F_t$ (kip) | $F_L$ (kip) | $F_v$ (kip) | $L_t/L_L$ (ft) | $L_v$ (ft) | $H_e$ (in) | $H_{min}$ (in) |
|---|---|---|---|---|---|---|---|---|
| TL-1 | 18 or above | 13.5 | 4.5 | 4.5 | 4.0 | 18.0 | 18.0 | 18.0 |
| TL-2 | 18 or above | 27.0 | 9.0 | 4.5 | 4.0 | 18.0 | 20.0 | 18.0 |
| TL-3 | 29 or above | 71.0 | 18.0 | 4.5 | 4.0 | 18.0 | 19.0 | 29.0 |
| TL-4 (a) | 36 | 68.0 | 22.0 | 38.0 | 4.0 | 18.0 | 25.0 | 36.0 |
| TL-4 (b) | between 36 and 42 | 80.0 | 27.0 | 22.0 | 5.0 | 18.0 | 30.0 | 36.0 |
| TL-5 (a) | 42 | 160.0 | 41.0 | 80.0 | 10.0 | 40.0 | 35.0 | 42.0 |
| TL-5 (b) | greater than 42 | 262.0 | 75.0 | 160.0 | 10.0 | 40.0 | 43.0 | 42.0 |
| TL 6 | | 175.0 | 58.0 | 80.0 | 8.0 | 40.0 | 56.0 | 90.0 |

References:

- TL-1 and TL-2 Design Forces are from AASHTO LRFD Section 13 Table A13.2-1
- TL-3 Design Forces are from research conducted under NCHRP Project 20-07 Task 395
- TL-4 (a), TL-4 (b), TL-5 (a), and TL-5 (b) Design Forces are from research conducted under NCHRP Project 22-20(2)

## CONCRETE PARAPET CAPACITY

*1. Determine $M_W$: flexural resistance of the parapet about its vertical axis. Positive and negative moment strength must be evaluated but will be equal based on barrier longitudinal reinforcement.*

| | | | | |
|---|---|---|---|---|
| Back face horizontal reinforcement | Size = **# 3** | Bar Diameter = | 0.375 | in. |
| | Number of bars = **2** | Bar Area = | 0.11 | $in^2$ |
| | | Stirrup Dia. = | 0.375 | in. |
| | | Design strip, b = | 13.0 | in. |

Area of steel per design strip — $A_S$ = Bar Area * NO. of bars = 0.22 $in.^2$/ft.

Effective depth of section — $d_S$ = d - c - Stirrup Dia. - 1/2 Bar Dia. = 15.44 in.

Depth of equivalent stress block

$$a = \frac{A_S f_y}{0.85 f_c' b} =$$ 0.22 in. * 75.0 ksi / (0.85 * 4.50 ksi * 13.0 in.) = 0.332 in.

Flexural resistance $$M_W = \varphi_{EE} A_S f_y \left(d_S - \frac{a}{2}\right) =$$

1.0 * 0.22 in. * 75.0 ksi * (15.44 in. - 0.33 in. / 2) / 12 in./ft. = 21.00 kip-ft.

*2. Determine $M_C$: flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge. Flexural moment resistance is based on the vertical reinforcement in the barrier.*

| | | | | |
|---|---|---|---|---|
| Stirrup Size = | **# 3** | Bar Diameter = | 0.375 | in. |
| Stirrup spacing = | **6.00 in.** | Bar Area = | 0.11 | $in^2$ |

Area of steel per design strip — $A_S$ = Bar Area * b / Stirrup spacing = 0.22 $in.^2$/ft.

Effective depth of section — $d_S$ = d - c - 1/2 Stirrup Dia. = 15.81 in.

Depth of equivalent stress block $$a = \frac{A_S f_y}{0.85 f_c' b} =$$ 0.36 in.

Flexural moment resistance $$M_c = \varphi_{EE} A_S f_y \left(d_S - \frac{a}{2}\right) =$$ 21.50 kip-ft./ft.

Critical length of yield line failure pattern $L_C = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H_W(M_b + M_W)}{M_C}} =$ 6.37 ft.

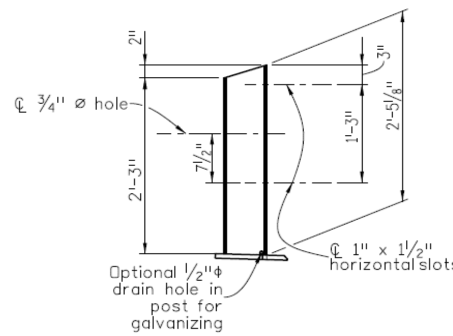
There is no additional resistance at the top of the parapet in addition to $M_W$ , $M_b$ = 0 kip-ft.

*3. Determine $R_W$ (nominal railing resistance to transverse load) within a wall segment.*

$$R_W = \left(\frac{2}{2L_C - L_t}\right)\left(8M_b + 8M_W + \frac{M_C L_C^2}{H_W}\right) =$$

244.67 kip AASHTO A13.3.1-1

*4. Calculate maximum post capacity $P_P$.*



**POST ELEVATION**

a. Plastic moment capacity of the post

| | | | |
|---|---|---|---|
| Yielding of post | $M_{post}$= | 62.08 | kip-ft |
| CG of impact force above curb | $H_R$ - $H_W$ | 19.06 | in |

Maximum shear force at base of the post, $P_p$ to cause post failure

| | | | |
|---|---|---|---|
| $M_{post}$ / ($H_R$ - $H_W$) | $P_{p1}$= | 39.08 | kip |

b. Weld connection strength

| | | | |
|---|---|---|---|
| Thickness of the weld | $t_{weld}$= | 0.313 | in |
| Effective thickness 0.77*$t_{weld}$ | $t_{weff}$= | 0.22 | in |

Calculate fillet weld strength as a line (Design of Welded Structures by Blodgett)

| | | | | |
|---|---|---|---|---|
| $S_W$ =(2*b*d + $\frac{d^2}{3}$)*$t_{weff}$ | $S_W$ = | 19.32 | $in^3$ | |
| Strength of the weld | $F_{EXX}$ = | 70.00 | ksi | |
| Maximum weld moment | $M_{weld}$ = | 67.63 | kip-ft | (0.6 * $F_{EXX}$ * $S_W$) |
| Maximum shear force at base | $P_{p2}$= | 42.58 | kip | |

c. Bolt shear strength

Shear resistance $R_n = 0.45 A_b F_{ub} N_s$ AASHTO 6.13.2.7-2

$R_n$ = 0.45 * (pi * 7/8 in ^2)/4 * 120.0 ksi * 2 $P_{p3}$= 64.94 kip

d. Concrete breakout shear strength

Spacing of bolts $b_{spa}$ = **9.00** in ACI 318 17.7.2

Since the spacing of the anchors is less than 3 times the bolt distance $d_b$, the bolts must be treated as a group

Area resisting breakout $A_{VC}$ = 585 $in^2$ (9.0 in + 3 * 10.0 in) * 1.5 * 10.0 in

Maximum area = $n_b$ * 4.5 $d_{bo}^2$ 900 $in^2$ $A_{VCO}$ = 450 $in^2$

$$V_{cb} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b$$

There is no eccentricity in shear loading and so modification factor for eccentricity $\psi_{ec,V}$ = 1.0 ACI 318 17.7.2.3

Edge distances (along the curb) > 1.5 x bolt distance and so modification factor for edge distance $\psi_{ed,V}$ = 1.0 ACI 318 17.7.2.4

Analysis indicates no cracking at service loads and so modification factor for concrete $\psi_{c,V}$ = 1.4 ACI 318 17.7.2.5

| | | |
|---|---|---|
| Anchor embedment $h_{ef}$ = | 10.75 | in |
| 1.5 * $d_{bo}$ = | 15.00 | in |

$$\psi_{h,V} = \sqrt{\frac{1.5 d_b}{h_{ef}}}$$ 1.181 ACI 318 17.7.2.6

Basic shear strength is minimum of $V_{b1} = \left(7\left(\frac{l_e}{\varphi}\right)^{0.2}\sqrt{\varphi}\right)\lambda_a\sqrt{f'_c}(d_{bo})^{1.5}$ $V_{b2} = 9\lambda_a\sqrt{f'_c}(d_{bo})^{1.5}$

$V_{b1}$ = 21.05 kip $V_{b2}$ = 19.09 kip

Load bearing length in shear $l_e$ = 7 in (Min of $h_{ef}$ and $8\phi$)
$\lambda_a$ = 1.0 for normal weight concrete
Basic shear strength $V_b$ = 19.09 kip

Shear strength $P_{p4}$= 41.05 kip
e. Bolt tensile strength (Ignore self weight)

$$\phi N_{sa} = \phi A_{se,N} f_{uta} \qquad A_{se} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2$$

Bolt tensile strength $f_{uta}$ = 120.0 ksi Bolt outside diameter $d_a$ = 0.895 in
$\phi$= 0.75 Number of threads/in. $n_t$ = 9 in
$A_{se}$ = 0.486 $in^2$
$N_{sa}$ = 43.75 kip

Tensile strength of 2 bolts = $N_s$ = 87.50 kip
Equating tension and compression, depth of compression c = $N_s$ / (0.85 * f'c * $W_b$) c = 1.91 in
Moment lever arm = 7" - c/2 6.05 in
Moment capacity based on bolt tensile capacity $M_{bolt}$ = 44.09 kip-ft
$M_{bolt}$ / ($H_R$ - $H_W$) $P_{p5}$= 27.76 kip

Minimum strength of post in shear $P_P$ = **27.76 kip**

4. Calculate collision tensile force in deck T and collision moment $M_{CT}$.

The resistance of each component of a combination bridge rail shall be determined as specified in Article A13.3.1 and A13.3.2 of the AASHTO code. The flexural strength of the rail shall be determined over one and two spans. The resistance of the combination parapet and rail shall be taken as the lesser of the resistances determined for the two failure modes.

Impact at Midspan (3 spans) (Other odd spans didn't control and so not included)
Number of spans N= 3

Yielding of all rails $M_p$= 93.33 kip-ft
Impact force distribution $L_t$ = 5.00 ft
post spacing L= 10.00 ft

AASHTO A13.3.2-1

$$R_R = \frac{16M_p + (N-1)(N+1)P_pL}{2NL - L_t}$$

=(16 * 93.33 kip-ft + 0) / (2 * 3 ft * 10.00 ft - 5.00 ft)
$R_R$= 67.53 kip

$$\bar{R} = R_R + R_w$$

=(67.53 kip +244.67 kip) = 312.1985 kip AASHTO A13.3.3-1

Designing deck overhang for strength > strength of rails and curb is conservative. Therefore, design only for maximum MASH $F_t$ loads. Assume the rails fail during impact and curb resists the remaining load.

Therefore Use $R_W$= 12.47 kip (80.00 kip - 67.53 kip ) Single span $\bar{R}$ = 80.00 kip
AASHTO A13.3.3-2

$$\bar{Y} = \frac{R_R H_R + R_w H_w}{\bar{R}}$$

=(67.53 kip * 32.50 in. +12.47 kip * 13.44 in.) / 80.00 kip
Y= 29.53 in

$$T = \frac{R_w}{L_C + 2H_w}$$

$T_{mid}$ = 1.45 kip/ft

$$M_{CT} = T * H_w$$

$M_{CTmid}$ = 1.62 kip-ft/ft

Impact at Post (2 spans) (Other even spans didn't control and so not included)
Number of spans N= 2

Impact force distribution $L_t$ = 5.00 ft
post spacing L= 10.00 ft

AASHTO A13.3.2-2

$$R'_R = \frac{16M_p + N^2 P_p L}{2NL - L_t}$$

=(16 * 93.33 kip-ft + 2^2 * 27.76 * 10.00/ (2 * 2 * 10.00 ft - 5.00 ft)

$R'_R$= 74.39 kip

$$R'_w = \frac{R_w H_w - P_p H_R}{H_w}$$

=(244.67 kip * 13.44 in. -27.76 kip * 32.50 in.) / 13.44 in AASHTO A13.3.3-5

$R'_W$= 177.54 kip

$$\bar{R} = P_p + R'_R + R'_w$$

= 27.76 kip + 74.39 kip + 177.54 kip 279.69 kip HTO A13.3.3-3

Use $\bar{R} =$ 80 kip Ignore $R'_W$ and use reduced $R'_R$ = 52.24 kip (80.00 kip - 27.76 kip )

AASHTO A13.3.3-4

$$\bar{Y} = \frac{P_p H_R + R'_R H_R + R'_w H_w}{\bar{R}}$$

=(27.76 kip * 32.50 in. +52.24 kip * 32.50 in. + 0.00 kip * 13.44 in. ) / 80.00 kip

Y= 32.5 in

$$T = \frac{P_p}{W_b + d_b + 2H_w}$$

$T_{post}$ = 7.26 kip/ft

$$M_{CT} = T * \bar{Y}$$

$M_{CTpost}$ = 19.66 kip-ft/ft

Use greater of the two failure modes \ $M_{ct}$ = 19.66 kip-ft/ft T = 7.26 kip/ft

**SUMMARY**

Impact at post controls the design as the transfer width is narrower than the impact between posts

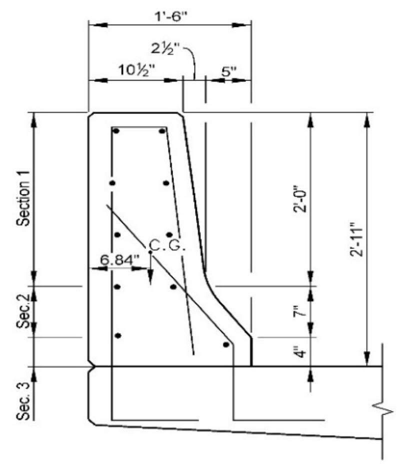
| | | | |
|---|---|---|---|
| Controlling Axial Load Per Unit Length of the Deck | $T_{Axial}$ = | 7.26 | kip/ft. |
| Deck Overhang Moment | $M_{ct}$ = | 19.66 | kip-ft./ft. |

# EXAMPLE 6.3 - BARRIER TYPE 7 STRENGTH DESIGN

## GENERAL INFORMATION

The CDOT Bridge Rail Type 7 design follows the AASHTO LRFD Bridge Design Specifications A13.3.1 design procedure for concrete railings, using strength design for reinforced concrete. Although the Type 7 is not an accepted bridge rail for new bridges (retired), the design methodology is similar to what would be done for the new Type 9 Bridge Rail. The following calculations show case of impact within barrier segment, assuming that barrier will be extended past the limits of the bridge. For cases concerning impact at end of the barrier, refer to AASHTO Appendix A13.

| | | | | |
|---|---|---|---|---|
| Overall barrier height | $H_B$ = | **35.00** | in. | |
| Concrete strength | $f'_c$ = | **4.50** | ksi (Concrete Class D compressive strength) | |
| Reinforcement strength | $f_y$ = | **60.00** | ksi (Specified minimum yield strength of grade 60 steel) | |
| Concrete cover | c = | **2.00** | in. | |
| Resistance factor | $\varphi$ = | **1.00** | (Extreme Event) | AASHTO 1.3.2.1 |
| Test level | | **TL-4** | | AASHTO T A13.2-1 |
| Transverse design force | $F_t$ = | **54.00** | kips | |
| Impact force distribution | $L_t$ = | **3.50** | ft. | |




**Barrier Type 7**

Barrier Dimensions

| | Section1 | Section2 | Section3 | |
|---|---|---|---|---|
| Section top width | **10.50** | **13.00** | **18.00** | in. |
| Section bottom width | **13.00** | **18.00** | **18.00** | in. |
| Section height | **24.00** | **7.00** | **4.00** | in. |

Center of gravity from back face $X_{C.G.}$ = **6.84** in.

## BARRIER FLEXURAL CAPACITY

*1. Determine $M_C$: flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge. Flexural moment resistance is based on the vertical reinforcement in the barrier.*

Front face vertical reinforcement: **# 4 @ 8.00 in**

Bar Diameter = 0.50 in.
Bar Area = 0.20 in.$^2$

| | $A_S$ (in.$^2$) | $h_{(avg)}$ (in.) | $d_S$ (in.) | b (in.) | k= .85f'$_C$b | a=$A_Sf_y$/k (in.) | $\varphi M_n$ (kip-ft.) | $M_C$ (kip-ft./ft.) |
|---|---|---|---|---|---|---|---|---|
| Section 1 | 0.30 | 11.75 | 9.50 | 12.00 | 45.90 | 0.39 | 13.96 | 9.57 |
| Section 2 | 0.30 | 15.50 | 13.25 | 12.00 | 45.90 | 0.39 | 19.58 | 3.92 |
| Section 3 | 0.30 | 18.00 | 15.75 | 12.00 | 45.90 | 0.39 | 23.33 | 2.67 |
| | | | | | | | Barrier $M_C$ = | 16.15 kip-ft./ft. |

- $A_S$ - area of steel per design strip
- $h_{(avg)}$ - average section width
- $d_S$ - effective depth of design section
- b - width of design strip
- a - depth of equivalent stress block

$$\varphi M_n = \varphi A_S f_y \left(d_S - \frac{a}{2}\right)$$

$$M_C = \sum_1^n \varphi M_n \cdot SectionHeight\ /H_B$$

*2. Determine $M_W$: flexural resistance of the parapet about its vertical axis.*

Front and back face horizontal reinforcement **# 4**

Bar Diameter = 0.50 in.
Bar Area = 0.20 $in^2$
Stirrup Dia. = **0.50** in.

| | No. of Bars | $A_S$ (in.$^2$) | $h_{(avg)}$ (in.) | $d_S$ (in.) | b (in.) | k= .85f'$_C$b | a=$A_S f_y$/k (in.) | $\varphi M_W$ (kip-ft.) |
|---|---|---|---|---|---|---|---|---|
| Section 1 | 3 | 0.60 | 11.75 | 9.00 | 24.00 | 91.80 | 0.39 | 26.41 |
| Section 2 | 1 | 0.20 | 15.50 | 12.75 | 7.00 | 26.78 | 0.45 | 12.53 |
| Section 3 | 1 | 0.20 | 18.00 | 15.25 | 4.00 | 15.30 | 0.78 | 14.86 |
| | | | | | | | Barrier $M_W$ = | 53.80 kip-ft. |

*3. Rail resistance within a wall segment.*

$$R_W = \left(\frac{2}{2L_C - L_t}\right)\left(8M_b + 8M_W + \frac{M_C L_C^2}{H}\right)$$ AASHTO A13.3.1-1

$$L_C = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_W)}{M_C}}$$ AASHTO A13.3.1-2

Additional flexural resistance at top of wall $M_b$ = 0.00 kip-ft.
Critical length of yield line $L_C$ = 10.74 ft.
Nominal transverse load resistance $R_W$ = 118.93 kips

Capacity Check Check $R_W > F_t$ : 118.93 > 54.00 **OK**

**BARRIER INTERFACE SHEAR CAPACITY** AASHTO 5.7.4

Evaluate the shear capacity of the cold joint to transfer nominal resistance $R_W$ between the deck and railing. Neglect effects of barrier Dead Load and assume that the surface of the deck is not roughened.

Interface width considered in shear transfer $b_V$= **18.00** in.
Interface length considered in shear transfer $L_V$ = **12.00** in.

Shear contact area $A_{CV} = b_V L_V$ = 216.00 in.$^2$

Shear reinforcement at front face **# 4 @ 8.00 in.** Bar Area = 0.2 in.$^2$

Area of shear reinforcement $A_{VF}$ = 12.0 in. * 0.20 in. / 8.00 in. = 0.3 in.$^2$/ft.

$$Check \quad A_{vf} \geq \frac{0.05A_{cv}}{f_y} =$$ 0.18 **OK** AASHTO 5.7.4.2-1

Permanent compression force from barrier weight (neglected) Pc = **0.00** kip

For concrete placed against clean concrete surface, free of laitance, but not intentionally roughened

Cohesion factor c = **0.075** ksi AASHTO 5.7.4.4
Friction factor μ = **0.6**
Shear factor 1 $K_1$ = **0.2** (Fraction of concrete strength available to resist interface shear)
Shear factor 2 $K_2$ = **0.8** ksi (Limiting interface shear resistance)

$$V_n = \min \begin{cases} K_1 f'_c A_{CV} = & 0.20 * 4.50 \text{ ksi} * 216.0 \text{ in.} = 194.4 \text{ kip} \\ K_2 A_{CV} = & 0.80 * 216.0 \text{ in.} = 172.8 \text{ kip} \\ cA_{CV} + \mu(A_{VF} f_y + P_C) = & 0.075 \text{ ksi}*216\text{in.}+0.60(0.30 \text{ in.}* 60 \text{ ksi}+0\text{kip}) = 27.00 \text{ kip} \end{cases}$$

AASHTO 5.7.4.3

| | | | | |
|---|---|---|---|---|
| Resistance factor | φ = | 1.00 | (Extreme Event) | AASHTO 1.3.2.1 |
| Factored Shear Resistance | φVn = | 27.00 | kip | |
| Shear force acting on the barrier per 1.00 ft. strip | | | $V_u = \frac{R_W}{L_C} =$ 11.08 kip/ft. | |

Capacity Check  Check $\varphi V_n > V_u$ :  27.00  >  11.08  **OK**

**OVERHANG DESIGN DATA**

Barrier Type 7 satisfies all checks outlined in AASHTO LRFD Bridge Design Specifications Appendix 13. Use the following data for Deck overhang design when Barrier Type 7 is used (Test Level 4):

$T_{Axial} = R_W /(L_C + 2H_B)$  AASHTO A13.4.2-1

| | | | |
|---|---|---|---|
| Axial Load Per Unit Length of the Deck | $T_{Axial}$ = | 7.18 | kip/ft. |
| Moment Capacity of the Barrier | $M_c$ = | 16.15 | kip-ft./ft. |

**Note:** This example does not use MASH loads.

# EXAMPLE 6.4 - OVERHANG DESIGN

## GENERAL INFORMATION

Bridge deck overhang shall be designed for three separate design cases: AASHTO A13.4.1

- Case 1 - Horizontal and longitudinal forces from vehicle collision load (Extreme Event II limit state)
- Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
- Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)

The deck overhang region shall be designed to have resistance larger than the MASH impact forces. Therefore, analysis of MASH barriers must be done. Refer to Example 6.2 for detailed strength calculations for Barrier Type 10 MASH.

| | | | | |
|---|---|---|---|---|
| Barrier type | | **Type 10MASH** | | |
| Width of barrier base | $W_B$ = | 18.0 | in. | |
| Barrier weight | $W_{Barrier}$ = | 0.461 | kip/ft. (see Deck Design) | |
| Deck overlay density | $W_{WS}$ = | 0.147 | kcf | Section 3.4.2 |
| Concrete density | $W_C$ = | 0.15 | kcf | |
| Barrier center of gravity | $X_{C.G.}$ = | 12.70 | in. | |
| Axial load per unit length | $T_{Axial}$ = | 7.26 | kip/ft. (refer to Type 10MASH Strength Design) | |
| Moment capacity of the barrier | $M_C$ = | 19.66 | kip-ft./ft. (refer to Type 10MASH Strength Design) | |
| Critical length of yield line | $L_C$ = | 6.37 | ft. (refer to Type 10MASH Strength Design) | |
| Overhang width | $S_{OH}$ = | 5.00 | ft. | |
| Edge of deck to edge of flange | $S_{Gdr\_Edge}$ = | 3.00 | ft. | |
| Overhang minimum depth | $t_{OH(min)}$ = | 8.00 | in. | |
| Overhang maximum depth | $t_{OH(max)}$ = | 10.00 | in. (at exterior edge of flange) | |
| Concrete top cover | $c_{Top}$ = | 2.00 | in. | AASHTO T.5.10.1-1 |
| Concrete strength | $f'_c$ = | 4.5 | ksi | |
| Reinforcement strength | $f_y$ = | 60 | ksi | |
| Test Level | | TL-4 | | |
| Transverse design force | $F_t$ = | 80 | kips | |
| Impact force distribution | $L_t$ = | 5 | ft | |
| Vertical Design Force | $F_V$ = | 22 | kips | |
| Longitudinal distribution of Vertical force | $L_V$ = | 18 | ft | |

| Controlling Load Combinations | Load Factors | | | |
|---|---|---|---|---|
| | $\gamma_{DC}$ | $\gamma_{DW\_max}$ | $\gamma_{CT}$ | $\gamma_{LL}$ |
| Extreme Event II | 1.00 | 1.00 | 1.00 | 0.50 |
| Strength I | 1.25 | 1.50 | 0.00 | 1.75 |

AASHTO T3.4.1-1

The deck overhang is designed to resist an axial tension force and moment from vehicular collision (CT) acting simultaneously with the Dead Load (DC/DW) and Live Load (LL) moment. The critical section shall be taken at the face of the box girder (AASHTO 4.6.2.1.6). In addition, Extreme Event II combination is also checked at the face of the curb. Loads are be assumed to be distributed at a 45 degree angle starting from the base plate.

## DESIGN CASE 1: Extreme Event II (Transverse Collision) at the face of the curb

Distance from edge of deck to design section $K$ = 1.50 ft. AASHTO 4.6.2.1.6

Distance from barrier face to design section $X$ = 0.00 ft.

Depth of the section under consideration $h_{Design}$ = 9.00 in. (may add min haunch depth if needed, conservative to use constant deck depth)

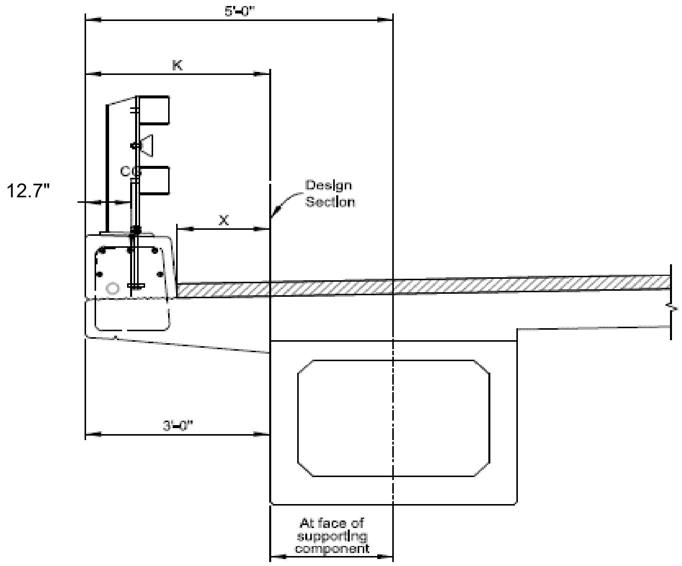
Bending moments from dead load of structural components and nonstructural attachments:

Barrier $M_{DC\text{-}Barrier} = W_{Barrier} * (K - X_{C.G.})$ = 0.461 kip/ft. * (1.50 ft. - 12.70 in. / 12 in./ft.) = 0.204 kip-ft./ft.

Deck $M_{DC\text{-}Deck} = W_C * t_{OH(min)} * K^2 / 2$ = 0.150 kcf * 8 in. / 12 in./ft. * (1.50 ft.) / 2 = 0.113 kip-ft./ft.

Additional overhang concrete $M_{DC\text{-}Add} = 0.5\ W_C * S_{Gdr\_Edge} (T_{OH(max)} - T_{OH(min)}) * (K - 2/3\ S_{Gdr\_Edge})$ =
= 0.5 * 0.150 kcf * 1.50 ft. * (10.0 in. - 8.0 in.) / 12 in./ft. * (1.50 ft. - 2/3 * 1.50 ft.) = 0.009 kip-ft./ft.

Total DC = $M_{DC\text{-}Barrier} + M_{DC\text{-}Deck} + M_{DC\text{-}Add}$ = 0.20 kip-ft.+0.11 kip-ft.+0.009 kip-ft. = 0.325 kip-ft./ft.




Bending moments from wearing surfaces and utilities:

Deck overlay $M_{DW\text{-}WS} = W_{WS} * 3\text{ in.} * X^2 / 2 =$ 0.147 kcf * 3in. / 12 in./ft. * (0.00 ft.) / 2 = 0.000 kip-ft./ft.

Both design bending moment and design axial tension are calculated based on the properties of the barrier on the deck. See Type 10MASH tab for information on strength design.

Bending moment from vehicular collision $M_{CT} = M_C =$ 19.66 kip-ft./ft.

Design factored moment (Extreme Event II, Case I) AASHTO 3.4.1, A13.4.1

$Mu_1 = 1.0M_{DC} + 1.0M_{DW} + 1.0M_{CT} =$ 0.325 kip-ft. + 0.000 kip-ft. + 19.66 kip-ft. = 19.99 kip-ft./ft.

**DESIGN CASE 2: Extreme Event II (Vertical Collision) at the face of the curb**

Vertical and Longitudinal collision cases will not control generally and so other critical sections are not included.

| | | | |
|---|---|---|---|
| Lever arm for vertical collision | $l_a$= | 0.442 | ft |
| Vertical Design Force | $F_V$ = | 22.00 | kips |
| Longitudinal distribution of Vertical force | $L_V$ = | 18.00 | ft |

Bending moment on overhang due to vertical forces

$M_{V\text{-}CT} = F_V * l_a / L_V =$ 22 kip * 0.44 ft. / 18.00 ft. = 0.540 kip-ft./ft.

Dead Load moment

$M_{DC} =$ 0.33 kip-ft./ft.

Design factored moment (Extreme Event II, Case I) AASHTO 3.4.1, A13.4.1

$Mu_2 = 1.0M_{DC} + 1.0M_{CT} =$ 0.540 kip-ft./ft. + 0.325 kip-ft./ft. = 0.866 kip-ft./ft.

**DESIGN CASE 3: STRENGTH I (At the face of the girder)**

The overhang is designed to resist gravity forces from the Dead Load of structural components and attachments to the cantilever, as well as a concentrated Live Load positioned 12.00 in. from the face of the barrier.

For decks with overhangs not exceeding 6.00 ft. measured from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity per AASHTO LRFD Bridge Design Specifications 3.6.1.3.4.

| | | | | |
|---|---|---|---|---|
| Distance from edge of deck to design section | K = | 3 | ft. | |
| Distance from barrier face to design section | X = | 1.5 | ft. | |
| Depth of the section under consideration | $h_{Design}$ = | 10.00 | in. | |
| Distance from LL application to design section | z = | 0.5 | ft. | |
| Live Load multiple presence factor | m = | 1.00 | | AASHTO T.3.6.1.1.2-1 |
| Dynamic load allowance | IM = | 0.33 | | AASHTO 3.6.2 |

Bending moment from Dead Loads (equal to the loads calculated for Design Case 1)

| | | | |
|---|---|---|---|
| Barrier | $M_{DC\text{-}Barrier}$ = | 0.894 | kip-ft./ft. |
| Deck | $M_{DC\text{-}Deck}$ = | 0.45 | kip-ft./ft. |
| Add. overhang concrete | $M_{DC\text{-}Add}$ = | 0.038 | kip-ft./ft. |
| Deck overlay | $M_{DW\text{-}WS}$ = | 0.041 | kip-ft./ft. |

AASHTO 3.6.1.3.4

Bending moment from live load $M_{LL}$ = 1.0 klf * 0.50 ft. = 0.5 kip-ft./ft.

Design factored moment (Strength I) $Mu_3 = 1.25M_{DC}+1.50M_{DW}+1.75m(M_{LL}+IM)$ =

= 1.25 * 1.38 kip-ft./ft + 1.50 * 0.041 kip-ft./ft + 1.75 * 1.00 * 1.33 * 0.50 kip-ft./ft = 2.95 kip-ft./ft.

<u>Design Summary</u> (By observation, other load cases will not control and are not included in this example)

| | | | | |
|---|---|---|---|---|
| Design Case 1 | $M_{u1}$ = | 19.990 | kip-ft./ft. | |
| Design Case 2 | $M_{u2}$ = | 0.866 | kip-ft./ft. | |
| Design Case 3 | $M_{u3}$ = | 2.953 | kip-ft./ft. | |
| Controlling Case = | Mu1 = | 19.990 | kip-ft./ft. | **DESIGN CASE 1 CONTROLS** |

Design axial tensile load $T_{Axial}$ = 7.26 kip/ft.

Top transverse reinforcement: Bar size **# 5** (see Deck Design)
Bar spacing s = **5** in.

Bottom transverse reinforcement: Bar size **# 5** (see Deck Design)
Bar spacing s = **6** in.

Area of top steel per design strip $A_{St} = b\ (A_b / s)$ = 12 in. * 0.31 in. / 5.0 in. = 0.744 in.$^2$/ft.

Area of bottom steel per design strip $A_{Sb} = b\ (A_b / s)$ = 12 in. * 0.31 in. / 5.0 in. = 0.62 in.$^2$/ft.

Steel in each layer resisting tension $A_{ten} = T_{axial} * 0.5 / F_y$ = 7.26 kip * 0.5 / 60.0 ksi = 0.061 in.$^2$/ft.

Area of top steel per design strip resisting moment $A_{st} - A_{ten}$
0.74 sq. in. - 0.06 sq. in. = 0.683 in.$^2$/ft.

Effective depth of section $d_S = h_{Design} - c_{Top} - 1/2\ d_b$ =
9 in. - 2 in. - 0.625 in./ 2 = 6.688 in.

Depth of equivalent stress block

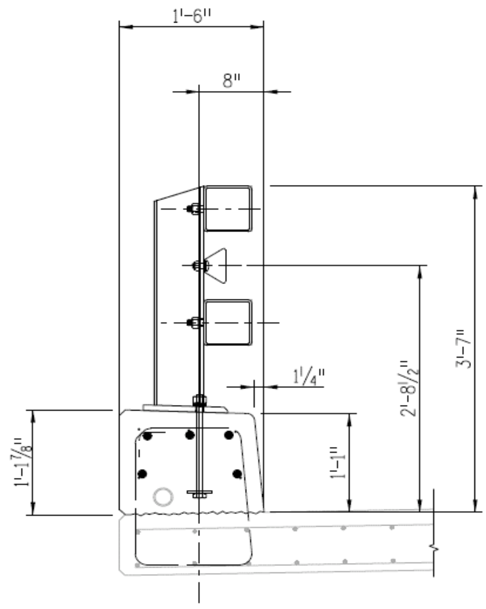
$$a = \frac{As\ * fy}{0.85 f_c' b} =$$ 0.56 sq. in. * 60 ksi / (0.85 * 4.50 ksi * 12 in.) = 0.893 in.

Factored flexural resistance $$\varphi_{EE} M_n = \varphi_{EE}\left[As\ * fy\left(d - \frac{a}{2}\right)\right] =$$

1.0 * 0.68 sq. in. * 60.00 ksi * ( 6.69 in. - 0.89 in / 2 ) = 21.328 kip-ft./ft.

21.328 > 19.990 **OK**

# BARRIER TYPE 10MASH CENTER OF GRAVITY




| Description | Unit wt lb/ft | Distance from deck out (in.) | Length (ft) | Number | Weight lb | Wx lb-in. |
|---|---|---|---|---|---|---|
| Tubes 6 x 6 x 1/4 | 19.02 | 13.61 | 10.00 | 2 | 380.40 | 5175.34 |
| Post W6 x 20 | 20.00 | 7.61 | 2.33 | 1 | 46.60 | 354.63 |
| Base Pl 10.5 x 12 x 11/16 | 24.56 | 8.25 | 1.00 | 1 | 24.56 | 202.65 |
| | | | | Total | **451.56** | **5732.62** |

CG from deck out = **12.70** in.