

APPENDIX A

EXAMPLE 6 - DECK DESIGN INCLUDING TYPE 10 MASH RAIL COLLISION

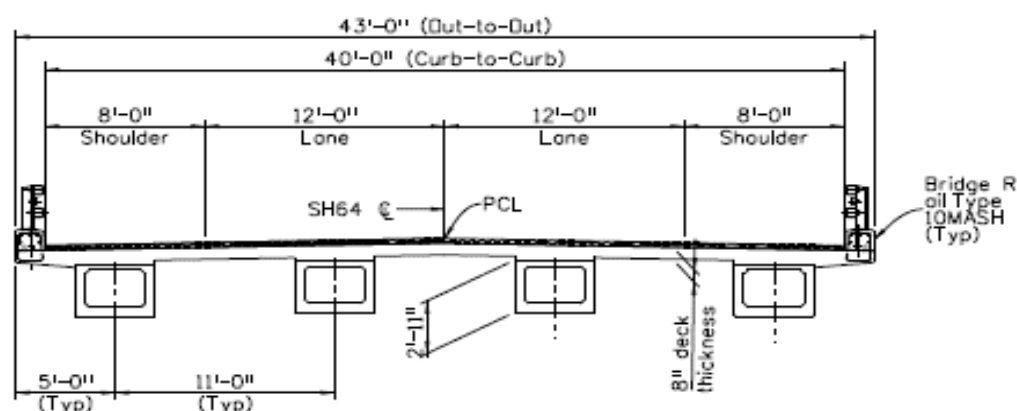
EXAMPLE 6.1 - DECK DESIGN

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
Concrete bottom cover	$C_{Bot} =$	1.0	in
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
Allowance for future utilities	$W_{util} =$	0.005	ksf
Resistance factors	$\phi_{STR} =$	0.9	(strength limit state)
	$\phi_{EE} =$	1.0	(extreme event limit state)
Correction factor for source aggregate	$K_1 =$	1	
Modulus of elasticity of reinforcement	$E_s =$	29000.0	ksi
Modulus of elasticity of concrete	$E_c =$	4435.3	ksi
$E_c = 120,000 K_1 W_c^2 f'_c{}^{0.33}$			
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	$A_B =$	0.059	CY/ft
Barrier Weight	$W_{\text{barrier}} =$	0.289	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 wl^2 , $+M = 0.080$ 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

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

Positive Moment

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

Negative Moment

$-M_{\text{deck}} =$	0.100 klf * (11.00 ft) ² * 0.10 =	1.21 k-ft/ft
$-M_{\text{WS}} =$	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

γ - 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 Combination	Load Factors			Design Moments	
	γ_{DC_max}	γ_{DW_max}	γ_{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	$+M_u =$	14.80 kip-ft/ft
Controlling negative factored moment	$-M_u =$	-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 $\phi_{STR} = 0.9$ 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

$$A_s = b (A_b / s) = 12.0 \text{ in.} * 0.310 \text{ in.}^2 / 6.0 \text{ in.} = 0.62 \text{ in.}^2$$

Effective depth of section

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

Depth of equivalent stress block

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

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

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

$$Check + \phi M_n > +M_u: \quad 17.53 > 14.80 \quad \text{OK}$$

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

$$A_s = B (A_b / s) = 12 \text{ in.} * 0.310 \text{ in.}^2 / 5.00 \text{ in.} = 0.74 \text{ in.}^2$$

Effective depth of section

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

Depth of equivalent stress block

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

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

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

$$Check - \phi M_n > -M_u: \quad 17.41 > 10.09 \quad \text{OK}$$

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, $M_r = \phi M_n$, at least equal to the lesser of:

- 1.33 times the positive factored ultimate moment
- Cracking moment

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

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:

Flexural cracking variability factor

$$\gamma_1 = 1.6 \quad (\text{non-segmental brg.}) \quad \text{AASHTO 5.6.3.3}$$

Ratio of specified min. yield strength to ultimate tensile strength

$$\gamma_3 = 0.67 \quad (\text{A615 steel})$$

Concrete density modification factor

$$\lambda = 1.0 \quad \text{AASHTO 5.4.2.8}$$

$$\text{Modulus of rupture } f_r = 0.24 \lambda \sqrt{f'_c} = 0.509 \text{ ksi}$$

AASHTO 5.4.2.6

$$\text{Section modulus of design section } S_c = \frac{bh^2}{6} = \frac{bt_{deck}^2}{6} = 12.0 \text{ in.} \cdot (8.0 \text{ in.})^2 / 6 = 128 \text{ in.}^3$$

Check Positive Moment reinforcement

$$\text{Check } +\phi M_n \geq \min \left\{ \begin{array}{l} 1.33 (+M_u) = 1.33 \cdot 14.80 \text{ kip-ft.} = 19.68 \text{ kip-ft.} \\ M_{cr} = 0.67 \cdot 1.60 \cdot 0.51 \text{ ksi} \cdot 128.0 \text{ in.}^3 / 12 \text{ in./ft.} = 5.82 \text{ kip-ft.} \end{array} \right. \quad \text{OK}$$

Check Negative Moment reinforcement

$$\text{Check } -\phi M_n \geq \min \left\{ \begin{array}{l} 1.33 (-M_u) = 1.33 \cdot 10.09 \text{ kip-ft.} = 13.42 \text{ kip-ft.} \\ M_{cr} = 0.67 \cdot 1.60 \cdot 0.51 \text{ ksi} \cdot 128.0 \text{ in.}^3 / 12 \text{ in./ft.} = 5.82 \text{ kip-ft.} \end{array} \right. \quad \text{OK}$$

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.):

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

- In which:
- $\gamma_e = 1.00$ - exposure factor (1.0 for Class 1 and 0.75 for Class 2) (assume waterproofing membrane is used)
 - β_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 ($\leq 0.60 f_y$ 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):

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

$$\beta_s = 1 + \frac{d_c}{0.7(t_{Deck} - d_c)} = 1 + 1.31 \text{ in.} / [0.7(8.0 \text{ in.} - 1.31 \text{ in.})] = 1.28$$

$$\text{Tension reinforcement ratio } \rho = \frac{A_s}{bd_s} = 0.62 \text{ in.} / (12 \text{ in.} \cdot 6.69 \text{ in.}) = 0.008$$

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

$$j = 1 - k/3 = 0.910$$

$$f_{ss} = \frac{+M_{u_service}}{A_s j d_s} = 8.78 \text{ kip-ft.} \cdot 12 \text{ in./ft.} / (0.62 \text{ in.} \cdot 0.91 \cdot 6.69 \text{ in.}) = 27.95 \text{ ksi}$$

$$s_{max} = \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c = 700 \cdot 1.00 / (1.28 \cdot 27.95 \text{ ksi}) - 2 \cdot 1.31 \text{ in.} = 16.94 \text{ in.}$$

$$\text{Spacing of positive moment reinforcement used in the design} = 6.00 \text{ in.}$$

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

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

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

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

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

$$\text{Modular ratio } n = E_s / E_c = 29000 \text{ ksi} / 4435 \text{ ksi} = 6.54$$

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

$$j = 1 - k/3 = 0.896$$

$$f_{ss} = \frac{-M_{u_service}}{A_s j d_s} = 6.17 \text{ kip-ft.} * 12 \text{ in./ft.} / (0.74 \text{ in.}^2 * 0.90 * 5.69 \text{ in.}) = 19.55 \text{ ksi}$$

$$s_{max} = \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = 700 * 1.00 / (1.58 * 19.55 \text{ ksi}) - 2 * 2.31 \text{ in.} = 18.03 \text{ in.}$$

$$\text{Spacing of negative moment reinforcement used in the design} = 5.00 \text{ in.}$$

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

Check tensile stress at service limit state doesn't exceed $0.60f_y$

$$f_{ss} = 19.55 \text{ ksi}$$

$$0.60 f_y = 36 \text{ ksi} = 0.60 * 60 \text{ ksi}$$

$$\text{Check } f_{ss} \leq 0.60 f_y \quad \text{OK}$$

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:

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

$$0.11 \leq A_s \leq 0.60$$

AASHTO 5.10.6-2

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

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

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: $A_s = 0.40 \text{ in.}^2/\text{ft.}$
 $\text{Check } A_s \geq A_{s,min} \quad \text{OK}$

Effective span length $S = S_{\text{Gdr}} - \text{girder width} = 11.0 \text{ ft.} - 48.0 \text{ in.} / 12 \text{ in./ft.} = 7 \text{ ft.}$
 AASHTO 9.7.2.3

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

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

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

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

Bottom reinforcement try **# 5 @ 8.00 in.** on center: $A_s = 0.465 \text{ in.}^2/\text{ft.}$
 $\text{Check } A_s \geq A_{s,min} \quad \text{OK}$
 $\text{Check } A_s \geq A_{s_Req} \quad \text{OK}$

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.

The TL-4 maximum capacity of Type 10 MASH is shown for overhang example.

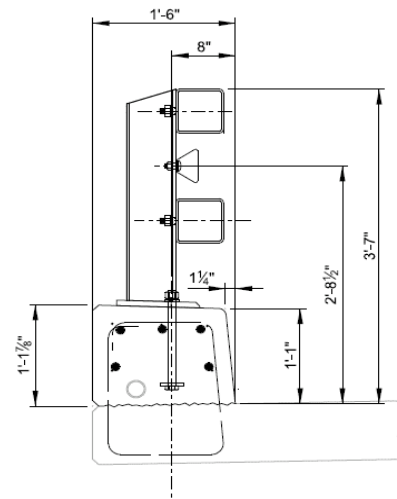
Overall barrier height	$H_B =$	43.0	in.	
Concrete cover (For SS rebars)	$c =$	1.5	in.	
Resistance factors	$\phi_{EE} =$	1	(Extreme Event)	AASHTO 1.3.2.1
	$\phi_S =$	0.8	(A325 bolts in shear)	AASHTO 6.5.4.2
	$\phi_T =$	0.8	(A325 bolts in tension)	AASHTO 6.5.4.2
Test level	MASH	TL-4		AASHTO T.A13.2-1
Transverse 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	$f_y =$	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 in.
Depth of W beam	$d_b =$ 6.2 in.
	F_y (post) = 50 ksi
	Z_{x-x} (post) = 14.9 in. ³
$M_n = M_p = F_y Z$ (F7-1 AISC Manual)	$M_{post} =$ 62.08 kip-ft



AISC Table 1-1

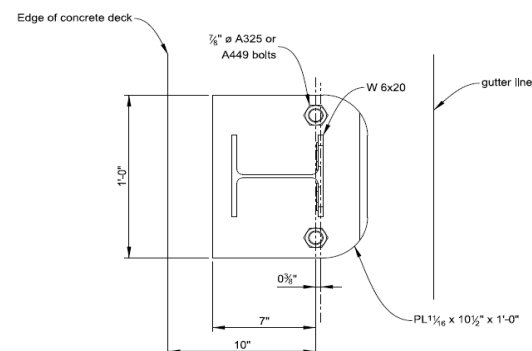
RAIL TUBES

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.
	F_y (tube) = 50.0 ksi
	Z (tube) = 11.2 in. ³
$M_n = M_p = F_y Z$ (F7-1 AISC Manual)	$M_p =$ 93.33 kip-ft

AISC Table 1-12

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	$\phi =$ 0.875 in.
Min tensile strength	$F_{ub} =$ 120.0 ksi
Number of bolts	$n_b =$ 2



The bar is hooked with a vertical embedment: **7** in Then the development fraction is:
 Development length factor = 0.82

Area of steel per design strip $A_s = \text{Bar Area} \times b / \text{Stirrup spacing} = 0.20 \text{ in.}^2/\text{ft.}$
 Effective depth of section $d_s = d - c - 1/2 \text{ Stirrup Dia.} = 16.25 \text{ in.}$

Depth of equivalent stress block $a = \frac{A_s f_y}{0.85 f'_c b} = 0.32 \text{ in.}$

Flexural moment resistance $M_c = \phi_{EE} A_s f_y \left(d_s - \frac{a}{2} \right) = 19.73 \text{ kip-ft./ft.}$

Critical length of yield line failure pattern $L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8 H_W (M_b + M_W)}{M_c}} = 7.38 \text{ ft.}$

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

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

$$R_W = \left(\frac{2}{2L_c - L_t} \right) \left(8M_b + 8M_W + \frac{M_c L_c^2}{H_W} \right) = 259.97 \text{ kip} \quad \text{AASHTO A13.3.1-1}$$

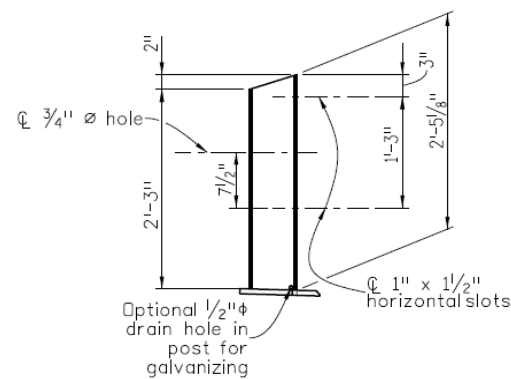
4. Calculate maximum post capacity P_p .

a. Plastic moment capacity of the post

Yielding of post $M_{\text{post}} = 62.08 \text{ kip-ft}$
 CG of impact force above curb $H_R - H_W = 19.06 \text{ in}$
 Maximum shear force at base of the post, P_p to cause post failure
 $M_{\text{post}} / (H_R - H_W) \quad P_{p1} = 39.08 \text{ kip}$

b. Weld connection strength

Thickness of the weld $t_{\text{weld}} = 0.313 \text{ in}$
 Effective thickness $0.77 t_{\text{weld}} \quad t_{\text{weff}} = 0.22 \text{ in}$



POST ELEVATION

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

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

c. Bolt shear strength

Shear resistance $R_n = 0.45 A_b F_{ub} N_s \quad \text{AASHTO 6.13.2.7-2}$
 $R_n = 0.45 \times (\pi \times 7/8 \text{ in}^2) / 4 \times 120.0 \text{ ksi} \times 2 = P_{p3} = 64.94 \text{ kip}$

d. Concrete breakout shear strength

Spacing of bolts $b_{\text{spa}} = 9.00 \text{ in} \quad \text{ACI 318 17.7.2}$

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 $A_{VC} = 585 \text{ in}^2 \quad (9.0 \text{ in} + 3 \times 10.0 \text{ in}) \times 1.5 \times 10.0 \text{ in}$
 Maximum area = $n_b \times 4.5 d_{bo}^2 \quad 900 \text{ in}^2 \quad A_{VCO} = 450 \text{ in}^2$

$$V_{cb} = \frac{A_{VC}}{A_{VCO}} \psi_{ec,v} \psi_{ed,v} \psi_{c,v} \psi_{h,v} V_b$$

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) $> 1.5 \times$ 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

$1.5 \times d_{bo} = 15.00$ in

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

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

Load bearing length in shear $l_e = 7$ in $V_{b1} = 21.05$ kip (Min of h_{ef} and 8ϕ) $V_{b2} = 19.09$ kip

$\lambda_a = 1.0$ for normal weight concrete

Basic shear strength $V_b = 19.09$ kip $\times 1.181 \times 1.4 \times A_{vc} / A_{vc0}$

Shear strength $P_{p4} = 41.05$ kip

e. Bolt tensile strength (Ignore self weight)

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

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
 $A_{se} = 0.486$ in²
 $N_{sa} = 43.75$ kip

Tensile strength of 2 bolts = $N_s = 87.50$ kip

Equating tension and compression, depth of compression $c = N_s / (0.85 \times f'_c \times W_b) = 1.91$ 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

$$R_R = \frac{16M_p + (N-1)(N+1)P_p L}{2NL - L_t} = (16 \times 93.33 \text{ kip-ft} + 0) / (2 \times 3 \text{ ft} \times 10.00 \text{ ft} - 5.00 \text{ ft}) \quad \text{AASHTO A13.3.2-1}$$

$$R_R = 67.53 \text{ kip}$$

$$\bar{R} = R_R + R_W = (67.53 \text{ kip} + 259.97 \text{ kip}) = 327.50036 \text{ kip} \quad \text{AASHTO A13.3.3-1}$$

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

$$\bar{Y} = \frac{R_R H_R + R_w H_w}{\bar{R}} = (67.53 \text{ kip} * 32.50 \text{ in.} + 12.47 \text{ kip} * 13.44 \text{ in.}) / 80.00 \text{ kip}$$

$$Y = 29.53 \text{ in}$$

$$T = \frac{R_w}{L_C + 2H_w} \quad T_{\text{mid}} = 1.30 \text{ kip/ft}$$

$$M_{CT} = T * H_w \quad M_{CT\text{mid}} = 1.45 \text{ kip-ft/ft}$$

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

Number of spans $N = 2$

Impact force distribution post spacing $L_t = 5.00 \text{ ft}$
 $L = 10.00 \text{ ft}$

$$R'_R = \frac{16M_p + N^2 P_p L}{2NL - L_t} = (16 * 93.33 \text{ kip-ft} + 2^2 * 27.76 * 10.00 / (2 * 2 * 10.00 \text{ ft} - 5.00 \text{ ft}))$$

$$R'_R = 74.39 \text{ kip} \quad \text{AASHTO A13.3.2-2}$$

$$R'_w = \frac{R_w H_w - P_p H_R}{H_w} = (259.97 \text{ kip} * 13.44 \text{ in.} - 27.76 \text{ kip} * 32.50 \text{ in.}) / 13.44 \text{ in.}$$

$$R'_w = 192.84 \text{ kip} \quad \text{AASHTO A13.3.3-5}$$

$$\bar{R} = P_p + R'_R + R'_w = 27.76 \text{ kip} + 74.39 \text{ kip} + 192.84 \text{ kip} = 294.99 \text{ kip} \quad \text{AASHTO A13.3.3-4}$$

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

$$\bar{Y} = \frac{P_p H_R + R'_R H_R + R'_w H_w}{\bar{R}} = (27.76 \text{ kip} * 32.50 \text{ in.} + 52.24 \text{ kip} * 32.50 \text{ in.} + 0.00 \text{ kip} * 13.44 \text{ in.}) / 80.00 \text{ kip}$$

$$Y = 32.5 \text{ in}$$

$$T = \frac{P_p}{W_b + d_b + 2H_w} \quad T_{\text{post}} = 7.26 \text{ kip/ft}$$

$$M_{CT} = T * \bar{Y} \quad M_{CT\text{post}} = 19.66 \text{ kip-ft/ft}$$

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

SUMMARY

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

Use the following data for Deck overhang design at the front face of the curb (Test Level 4):

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

EXAMPLE 6.3 - BARRIER TYPE 9 STRENGTH DESIGN

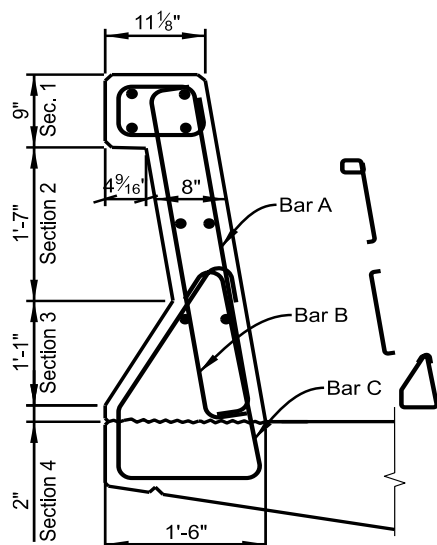
GENERAL INFORMATION

The CDOT Bridge Rail Type 9 design follows the AASHTO LRFD Bridge Design Specifications A13.3.1 design procedure for concrete railings, using strength design for reinforced concrete. 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. The applied design force (F_t) and the longitudinal length of distribution of the impact force (L_t) in this example is from the research conducted under NCHRP Project 22-20(2). **The TL-4 maximum capacity of Type 9 is shown for overhang example.**

Overall barrier height	$H_B =$	43.00	in.	
Concrete strength	$f'_c =$	4.50	ksi	
Reinforcement strength	$f_y =$	75.00	ksi	
Concrete cover	$c =$	1.50	in.	
Resistance factor	$\phi =$	1.00	(Extreme Event)	AASHTO 1.3.2.1
Test level		TL-4		AASHTO T A13.2-1
Transverse design force	$F_t =$	80.00	kips	
Impact force distribution	$L_t =$	5.00	ft.	

BARRIER FLEXURAL CAPACITY

1. Determine M_C : flexural resistance of cantilevered parapet about an axis parallel to the longitudinal axis of the bridge at midspan.



Barrier Dimensions	Sec. 1	Sec. 2	Sec. 3	Sec. 4	
Section top width	11.125	8.00	8.00	17.63	in.
Section bottom width	12.56	8.00	17.63	18.00	in.
Section height	9.00	19.00	13.00	2.00	in.

A_s - area of steel per design strip

h - section height

d_{avg} - average section width

d_s - effective depth of design section

b_c - width of design strip (taken as 1 ft per AASHTO Section 13)

a - depth of equivalent stress block

$$\phi M_n = \phi A_s f_y \left(d_s - \frac{a}{2} \right) \quad M_C = \sum_1^n \phi M_n / b_c$$

=====									
1st vertical rebar (Bar A)	# 4 @ 9 in.		Bar Diameter =		0.500 in.				
			Bar Area =		0.20 in. ²				
	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _c (kip-ft./ft.)
Section 1	0.27	9.00	11.84	10.09	12.00	45.90	0.44	16.46	3.45
Section 2	0.27	19.00	8.00	6.25	12.00	45.90	0.44	10.05	4.44
Section 3	0.27	13.00	12.81	11.06	12.00	45.90	0.44	18.07	5.46
Section 4	0.27	2.00	17.81	16.06	12.00	45.90	0.44	26.41	1.23
total H 43									Barrier M _c (Bar A) = 14.58

h/H x φM_n

.21 3.45

.44 4.44

.3 5.41

.047 1.24

2nd vertical rebar (Bar B)

4 @ 18 in.

	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _c (kip-ft./ft.)
Section 1	0.13	9.00	11.84	5.59	12.00	45.90	0.22	4.57	0.96
Section 2	0.00	19.00	8.00	1.75	12.00	45.90	0.00	0.00	0.00
Section 3	0.13	13.00	12.81	6.56	12.00	45.90	0.22	5.38	1.63
Section 4	0.13	2.00	17.81	11.56	12.00	45.90	0.22	9.54	0.44
Barrier M _c (Bar B) =									3.03

3rd vertical rebar (Bar C)

4 @ 12 in.

	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b _c (in.)	k = .85f' _c b	a = A _s f _y /k (in.)	φM _n (kip-ft.)	M _c (kip-ft./ft.)
Section 1	0.00	9.00	11.84	10.09	12.00	45.90	0.00	0.00	0.00
Section 2	0.00	19.00	8.00	6.25	12.00	45.90	0.00	0.00	0.00
Section 3	0.20	13.00	12.81	11.06	12.00	45.90	0.33	13.62	4.12
Section 4	0.20	2.00	17.81	16.06	12.00	45.90	0.33	19.87	0.92
Barrier M _c (Bar C) =									5.04

Grand Total Barrier M_c = 22.652. Determine M_w: flexural resistance of the parapet about its vertical axis.

Back face horizontal reinforcement

4

Bar Diameter = 0.50 in.

Bar Area = 0.20 in.²

	No. of Bars	A _s (in. ²)	h (in.)	d _{avg} (in.)	d _s (in.)	b (in.)	k = .85f' _c h	a = A _s f _y /k (in.)	φM _w (kip-ft.)
Section 1	2	0.40	9.00	11.84	9.59	9.00	34.43	0.87	22.90
Section 2	1	0.20	19.00	8.00	5.75	19.00	72.68	0.21	7.06
Section 3	1	0.20	13.00	9.63	7.38	13.00	49.73	0.30	9.03
Section 4	0	0.00	2.00	17.81	15.56	2.00	7.65	0.00	0.00
Barrier M _w =									38.98

3. Rail resistance within a wall segment.

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

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

Additional flexural resistance at top of wall	$M_b =$	0.00	kip-ft.
Critical length of yield line	$L_C =$	9.96	ft.
Nominal transverse load resistance	$R_W =$	125.86	kips

Capacity Check Check $R_W > F_t$: 125.86 > 80.00 **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.²Shear reinforcement at front face **# 4 @ 12 in.** Bar Area = 0.2 in.²Area of shear reinforcement $A_{VF} =$ 12 in. * 0.20 in. / 12 in. = 0.2 in.²/ft.

$$\text{Check } A_{vf} \geq \frac{0.05A_{cv}}{f_y} = 0.144 \quad \text{OK} \quad \text{AASHTO 5.7.4.2-1}$$

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

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

Cohesion factor $c =$ **0.075** ksi AASHTO 5.7.4.4Friction factor $\mu =$ **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)

$$V_n = \min \begin{cases} K_1 f'_c A_{CV} = 0.20 * 4.50 \text{ ksi} * 216.0 \text{ in.} = 194.4 \text{ kip} & \text{AASHTO 5.7.4.3} \\ K_2 A_{CV} = 0.80 * 216.0 \text{ in.} = 172.8 \text{ kip} \\ c A_{CV} + \mu (A_{VF} f_y + P_c) = 0.075 \text{ ksi} * 216 \text{ in.} + 0.60 (0.20 \text{ in.} * 75 \text{ ksi} + 0 \text{ kip}) = 25.20 \text{ kip} \end{cases}$$

Resistance factor $\phi =$ 1.00 (Extreme Event) AASHTO 1.3.2.1Factored Shear Resistance $\phi V_n =$ 25.20 kipShear force acting on the barrier per 1.00 ft. strip $V_u = \frac{R_W}{L_C} =$ 12.64 kip/ft.Capacity Check Check $\phi V_n > V_u$: 25.20 > 12.64 **OK**

OVERHANG DESIGN DATA

Barrier Type 9 satisfies all checks outlined in AASHTO LRFD Bridge Design Specifications Appendix 13. Use the following data for Deck overhang design at the front face of the curb when Barrier Type 9 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	$T_{Axial} =$	7.35	kip/ft.
Moment Capacity of the Barrier	$M_c =$	22.65	kip-ft./ft.

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	$W_{\text{Barrier}} =$	0.289	kip/ft. (see Deck Design)
Deck overlay density	$W_{\text{WS}} =$	0.147	kcf
Concrete density	$W_C =$	0.15	kcf
Barrier center of gravity	$X_{\text{C.G.}} =$	12.63	in.
Axial load per unit length	$T_{\text{Axial}} =$	7.26	kip/ft. (refer to Type 10MASH Strength Design)
Deck Overhang Moment	$M_C =$	19.66	kip-ft./ft. (refer to Type 10MASH Strength Design)
Critical length of yield line	$L_C =$	7.38	ft. (refer to Type 10MASH Strength Design)
Overhang width	$S_{\text{OH}} =$	5.00	ft.
Edge of deck to edge of flange	$S_{\text{Gdr_Edge}} =$	3.00	ft.
Overhang minimum depth	$t_{\text{OH(min)}} =$	8.00	in.
Overhang maximum depth	$t_{\text{OH(max)}} =$	10.00	in. (at exterior edge of flange)
Concrete top cover	$c_{\text{Top}} =$	2.00	in.
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

Section 3.4.2

AASHTO T.5.10.1-1

Controlling Load Combinations	Load Factors			
	Y_{DC}	$Y_{\text{DW_max}}$	Y_{CT}	Y_{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 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	$h_{\text{Design}} =$	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	$M_{\text{DC-Barrier}} = W_{\text{Barrier}} * (K - X_{\text{C.G.}}) =$	0.289 kip/ft. * (1.50 ft. - 12.63 in. / 12 in./ft.) =	0.129 kip-ft./ft.
Deck	$M_{\text{DC-Deck}} = W_C * t_{\text{OH(min)}} * K^2 / 2 =$	0.150 kcf * 8 in. / 12 in./ft. * (1.50 ft.) ² / 2 =	0.113 kip-ft./ft.

$$\begin{aligned} \text{Additional overhang concrete} \quad M_{\text{DC-Add}} &= 0.5 W_C * S_{\text{Gdr_Edge}} (T_{\text{OH(max)}} - T_{\text{OH(min)}}) * (K - 2/3 S_{\text{Gdr_Edge}}) = \\ &= 0.5 * 0.150 \text{ kcf} * 1.50 \text{ ft.} * (10.0 \text{ in.} - 8.0 \text{ in.}) / 12 \text{ in./ft.} * (1.50 \text{ ft.} - 2/3 * 1.50 \text{ ft.}) = \end{aligned}$$

0.009 kip-ft./ft.

$$\text{Total DC} = M_{\text{DC-Barrier}} + M_{\text{DC-Deck}} + M_{\text{DC-Add}} = 0.13 \text{ kip-ft.} + 0.11 \text{ kip-ft.} + 0.009 \text{ kip-ft.} = 0.251 \text{ kip-ft./ft.}$$

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	$h_{\text{Design}} =$	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	$M_{\text{DC-Barrier}} =$	0.562	kip-ft./ft.
Deck	$M_{\text{DC-Deck}} =$	0.45	kip-ft./ft.
Add. overhang concrete	$M_{\text{DC-Add}} =$	0.038	kip-ft./ft.
Deck overlay	$M_{\text{DW-WS}} =$	0.041	kip-ft./ft.

AASHTO 3.6.1.3.4

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

Design factored moment (Strength I) $M_{u3} = 1.25M_{\text{DC}} + 1.50M_{\text{DW}} + 1.75m(M_{\text{LL}} + \text{IM}) =$
 $= 1.25 * 1.05 \text{ kip-ft./ft} + 1.50 * 0.041 \text{ kip-ft./ft} + 1.75 * 1.00 * 1.33 * 0.50 \text{ kip-ft./ft} = 2.54 \text{ kip-ft./ft.}$

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

Design Case 1	$M_{u1} =$	19.916	kip-ft./ft.	
Design Case 2	$M_{u2} =$	0.798	kip-ft./ft.	
Design Case 3	$M_{u3} =$	2.538	kip-ft./ft.	
Controlling Case =	$M_{u1} =$	19.916	kip-ft./ft.	DESIGN CASE 1 CONTROLS

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

Top transverse reinforcement: Bar size **# 5** (see Deck Design)
 Bar spacing $s = 5 \text{ in.}$

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

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

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

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

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

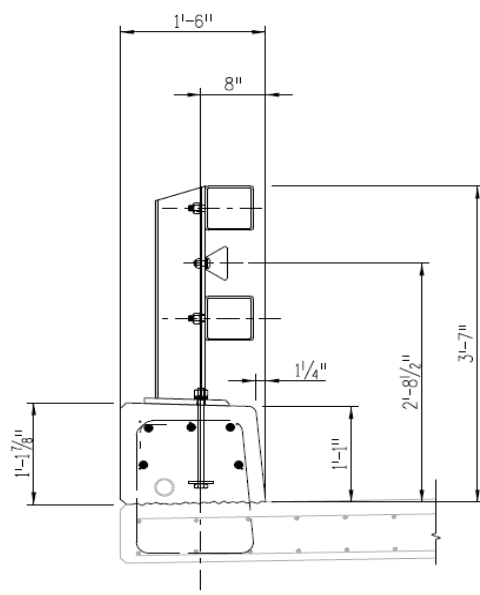
Effective depth of section $d_s = h_{\text{Design}} - c_{\text{Top}} - 1/2 d_b = 9 \text{ in.} - 2 \text{ in.} - 0.625 \text{ in.} / 2 = 6.688 \text{ in.}$

Depth of equivalent stress block

$$a = \frac{A_s * f_y}{0.85 f'_c b} = 0.68 \text{ sq. in.} * 60.00 \text{ ksi} / (0.85 * 4.50 \text{ ksi} * 12 \text{ in.}) = 0.893 \text{ in.}$$

Factored flexural resistance $\phi_{EE} M_n = \phi_{EE} \left[A_s * f_y \left(d - \frac{a}{2} \right) \right] =$
 $1.0 * 0.68 \text{ sq. in.} * 60.00 \text{ ksi} * (6.69 \text{ in.} - 0.89 \text{ in.} / 2) = 21.328 \text{ kip-ft./ft.}$
 21.328 > 19.916 **OK**

BARRIER TYPE 10 MASH CENTER OF GRAVITY (Steel Only)



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	13.50	10.00	2	380.40	5135.40
Post W6 x 20	20.00	7.4	2.339	1	46.78	346.17
Base PL 10.5 x 12 x 3/4	26.80	8.25	0.75	1	20.10	165.81
				Total	447.28	5647.38

CG from deck out = **12.63** in.