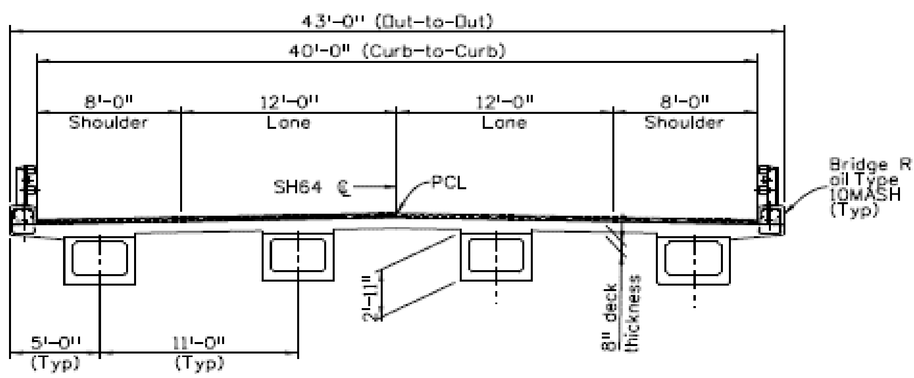
APPENDIX A
EXAMPLE 6 - DECK DESIGN INCLUDING TYPE 10 MASH RAIL COLLISION
EXAMPLE 6.1 - DECK DESIGN

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	
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	φ_{STR} =	**0.9**	(strength limit state)	
	φ_{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=	**0.059**	CY/ft
Barrier Weight	$W_{barrier}$=	**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 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).

$M_u = \eta\,[\gamma_{DC}M_{DC} + \gamma_{DW}M_{DW} + m\,\gamma_{LL}(M_{LL}+ IM)]$
η = 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 Factors			Design Moments	
Load Combination	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 +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	φ_{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 $in.^2$ * 60.0 ksi / (0.85 * 4.5 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 * 0.74$in.^2$* 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	γ_1 =	**1.6**	(non-segmental brg.)
Ratio of specified min. yield strength to ultimate tensile strength	γ_3 =	**0.67**	(A615 steel)
Concrete density modification factor	λ =	**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: γ_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.

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

Tension reinforcement ratio $\rho = \dfrac{A_S}{bd_S} =$ 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

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

$s_{max} = \dfrac{700\gamma_e}{\beta_S f_{SS}} - 2d_C =$ 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**

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

fss= 19.55 ksi

0.60 fy= 36 ksi = 0.60 * 60ksi

$Check\ f_{ss} \leq 0.60 f_y$ **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

$\dfrac{220}{\sqrt{S}} \leq 67\%$ $\dfrac{220}{\sqrt{S}} =$ 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

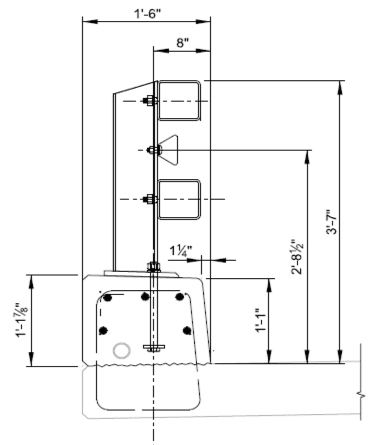
CDOT Bridge Rail Type 10MASH consists of a concrete parapet and a metal rail. The resistance to transverse vehicular impact loads shall be determined as specified in AASHTO LRFD Bridge Design Specifications A13.3.3. End impact is not considered. See CDOT Worksheet B-606-10MASH for barrier details.

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	φ_{EE} =	**1**	(Extreme Event)	AASHTO 1.3.2.1
	φ_S =	**0.8**	(A325 bolts in shear)	AASHTO 6.5.4.2
	φ_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	fy =	**75.0**	ksi



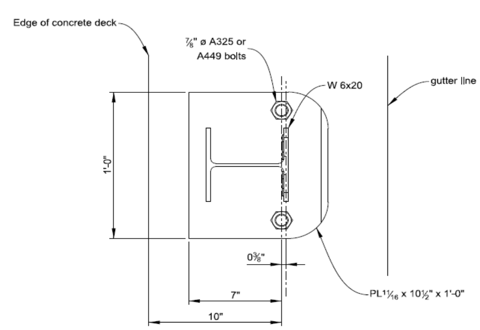
RAIL POST

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}	
Mn=Mp=FyZ (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	nTubes =	**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 = **# 4** — Bar Diameter = 0.5 in.

Number of bars = **2** — Bar Area = 0.2 in^2

Stirrup Dia. = 0.5 in.

Design strip, b = 13.0 in.

Area of steel per design strip — A_S = Bar Area * NO. of bars = 0.4 $in.^2/ft.$

Effective depth of section — d_S = d - c - Stirrup Dia. - 1/2 Bar Dia. = 15.75 in.

Depth of equivalent stress block

$$a = \frac{A_S f_y}{0.85 f_c' b} =$$ 0.40 in. * 75.0 ksi / (0.85 * 4.50 ksi * 13.0 in.) = 0.603 in.

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

1.0 * 0.40 in. * 75.0 ksi * (15.75 in. - 0.60 in. / 2) / 12 in./ft. = 38.62 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 = **# 4** — Bar Diameter = 0.5 in.

Stirrup spacing = **10.00 in.** Avg — Bar Area = 0.1635143 in^2

The straight bar development length is — Bar Area = 0.2 in^2

ldb = 23.00 in.

For a hooked # 4 bar, the basic development length lhb with modification factors is:

ldh= 11.20 in.

Therefore, the benefit derived from the hook is:

11.80 in

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 = Bar Area * b / Stirrup spacing = 0.20 in.^{2}/ft.

Effective depth of section d_S = d - c - 1/2 Stirrup Dia. = 16.25 in.

Depth of equivalent stress block $a = \dfrac{A_S f_y}{0.85 f_c' b} =$ 0.32 in.

Flexural moment resistance $M_c = \varphi_{EE} A_S f_y \left(d_S - \dfrac{a}{2}\right) =$ 19.73 kip-ft./ft.

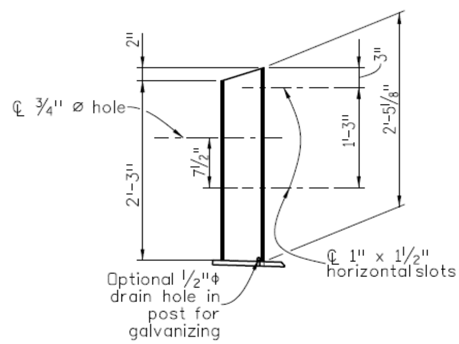
Critical length of yield line failure pattern $L_C = \dfrac{L_t}{2} + \sqrt{\left(\dfrac{L_t}{2}\right)^2 + \dfrac{8H_W(M_b + M_W)}{M_C}} =$ 7.38 ft.

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.

$$R_W = \left(\frac{2}{2L_C - L_t}\right)\left(8M_b + 8M_W + \frac{M_C L_C^2}{H_W}\right) =$$ **259.97** 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)

$S_W = (2*b*d + \dfrac{d^2}{3})*t_{weff}$ 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

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

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

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

Load bearing length in shear l_e = 7 in (Min of h_{ef} and 8ϕ)

λ_a = 1.0 for normal weight concrete

Basic shear strength V_b = 19.09 kip

Shear strength P_{p4}= 41.05 kip

<u>e. Bolt tensile strength</u> (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
	ϕ=	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.

<u>Impact at Midspan (3 spans)</u> (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 +259.97 kip) = 327.50036 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} = 1.30 kip/ft

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

M_{CTmid} = 1.45 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

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

=(259.97 kip * 13.44 in. -27.76 kip * 32.50 in.) / 13.44 in AASHTO A13.3.3-5

R'_W= 192.84 kip

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

= 27.76 kip + 74.39 kip + 192.84 kip 294.99 kip AASHTO A13.3

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
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_{Axial} =	7.26	kip/ft.
Deck Overhang Moment	M_{ct} =	19.66	kip-ft./ft.

EXAMPLE 6.3 - BARRIER TYPE 9 STRENGTH DESIGN

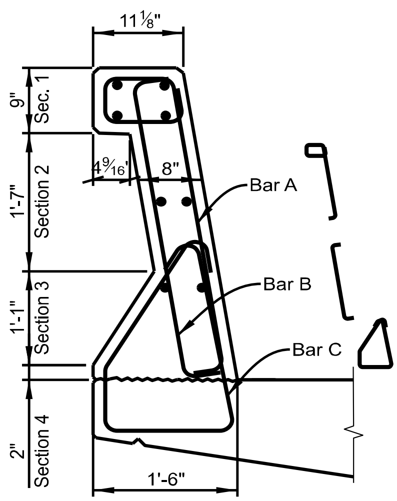
GENERAL INFORMATION

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	φ =	**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

$$\varphi M_n = \varphi A_S f_y \left(d_S - \frac{a}{2}\right) \qquad M_C = \sum_1^n \varphi M_n / \mathrm{b_c}$$

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= $.85f'_Cb$	$a=A_Sf_y/k$ (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
								Barrier M_C (Bar A) =	14.58

2nd vertical rebar (Bar B) **# 4 @ 18 in.**

	A_S ($in.^2$)	h (in.)	d_{avg} (in.)	d_S (in.)	b_c (in.)	k= $.85f'_Cb$	$a=A_Sf_y/k$ (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= $.85f'_Cb$	$a=A_Sf_y/k$ (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 = 22.65

2. 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= $.85f'_Ch$	$a=A_Sf_y/k$ (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.

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

Shear reinforcement at front face **# 4 @ 12 in.** Bar Area = 0.2 in.^{2}

Area of shear reinforcement A_{VF} = 12 in. * 0.20 in. / 12 in. = 0.2 in.^{2}/ft.

$$Check \quad A_{vf} \geq \frac{0.05A_{cv}}{f_y} =$$ 0.144 **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 of:

- $K_1 f'_c A_{CV} =$ 0.20 * 4.50 ksi * 216.0 in. = 194.4 kip — AASHTO 5.7.4.3
- $K_2 A_{CV} =$ 0.80 * 216.0 in. = 172.8 kip
- $cA_{CV} + \mu(A_{VF} f_y + P_C) =$ 0.075 ksi*216in.+0.60(0.20 in.* 75 ksi+0kip) = 25.20 kip

Resistance factor	φ =	1.00	(Extreme Event)	AASHTO 1.3.2.1
Factored Shear Resistance	φVn =	25.20	kip	

Shear force acting on the barrier per 1.00 ft. strip $V_u = \frac{R_W}{L_C} =$ 12.64 kip/ft.

Capacity Check — Check $\varphi V_n > V_u$: 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	$W_{Barrier}$ =	0.289	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	$X_{C.G.}$ =	12.63	in.	
Axial load per unit length	T_{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_{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			
	γ_{DC}	γ_{DW_max}	γ_{CT}	γ_{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:

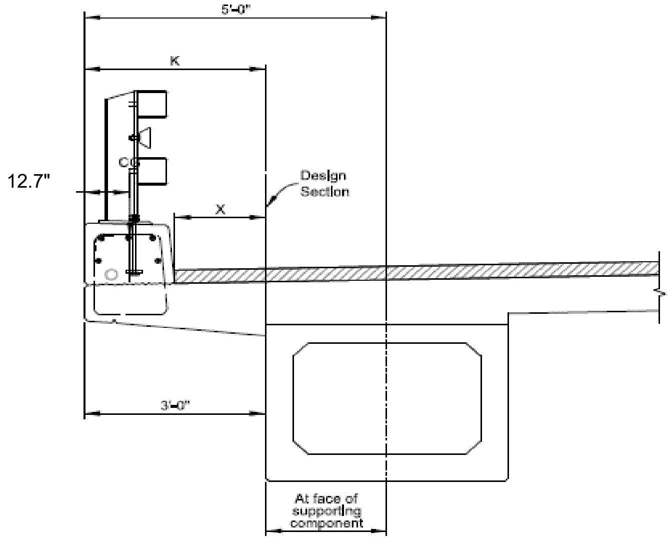
Barrier $M_{DC\text{-}Barrier} = W_{Barrier} * (K - X_{C.G.}) =$ 0.289 kip/ft. * (1.50 ft. - 12.63 in. / 12 in./ft.) = 0.129 kip-ft./ft.

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



Development length of transverse reinforcement should be considered. It can be dealt with in a variety of methods

Bending moments from wearing surfaces and utilities:

Deck overlay $M_{DW\text{-}WS} = W_{WS}$ * 3 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

$Mu_1 = 1.0M_{DC} + 1.0M_{DW} + 1.0M_{CT} =$ 0.251 kip-ft. + 0.000 kip-ft. + 19.66 kip-ft. = 19.92 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=	0.448	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 * 0.45 ft. / 18.00 ft. = 0.547 kip-ft./ft.

Dead Load moment

$M_{DC} =$ 0.25 kip-ft./ft.

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

$Mu_2 = 1.0M_{DC} + 1.0M_{CT} =$ 0.547 kip-ft./ft. + 0.251 kip-ft./ft. = 0.798 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	$M_{DC\text{-}Barrier}$ =	0.562	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 * 1.05 kip-ft./ft + 1.50 * 0.041 kip-ft./ft + 1.75 * 1.00 * 1.33 * 0.50 kip-ft./ft = 2.54 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} =	19.916	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 =	Mu1 =	19.916	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

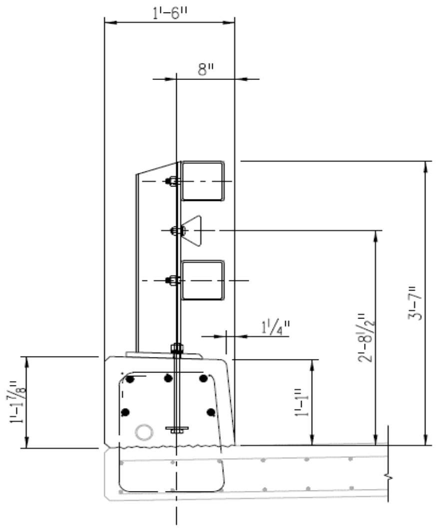
$$a = \frac{As\ * fy}{0.85 f_c' b} =$$ 0.68 sq. in. * 60.00 ksi / (0.85 * 4.50 ksi * 12 in.) = 0.893 in.

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 > 19.916 **OK**

BARRIER TYPE 10MASH 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.