EXAMPLE 12: RAIL ANCHOR SLAB DESIGN

General Information

Rail anchor slabs have been used with good performance under Allowable Strength Design (ASD) practices. AASHTO LRFD has since become the design standard and uses impact loads significantly larger than those in ASD. The LRFD impact loads result in a rail anchor slab that is much larger than what has proven successful in the past. NCHRP Report 663 concluded that AASHTO LRFD dynamic impact loads result in an overly conservative design for rail anchor slabs. New guidelines were established and validated through finite element modeling and full scale testing. NCHRP Report 663 recommends that a static load equivalent (L_s) of 10 kip be used to design rail anchor slabs for overturning and sliding design in lieu of AASHTO LRFD impact loads from Chapter 13. The static load equivalent of 10 kip is appropriate for designing rail anchor slabs for TL-4 test levels.

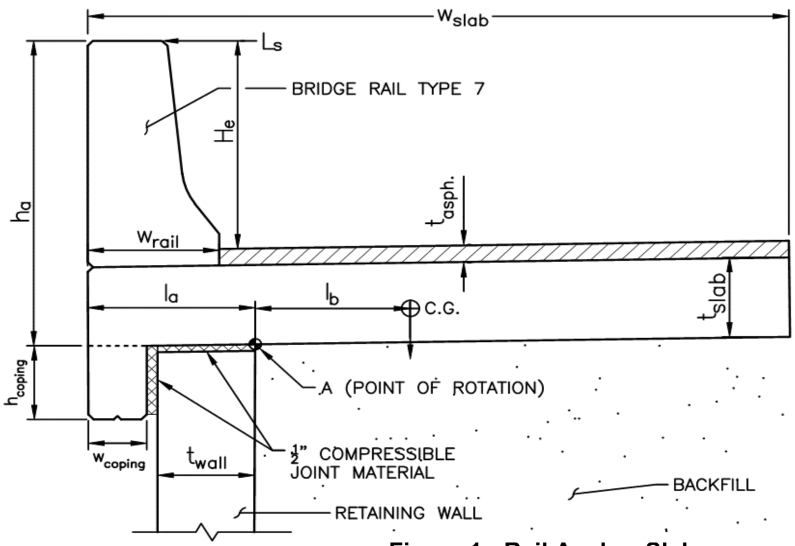

Figure 1 - Rail Anchor Slab

Rail Anchor Slab Inputs

This example illustrates the design of a rail anchor slab based on recommendations from NCHRP Report 663. Dimensions for this example are taken from CDOT Standard Sheet B-504-V1. Refer to this standard for additional details.

Concrete Unit Weight	$\gamma_{conc.}$ =	0.150	kcf	CDOT BDM 3.4.4.1
Asphalt Unit Weight	$\gamma_{asph.}$ =	0.147	kcf	CDOT BDM 3.4.2
Rail Anchor Slab Width	w_{slab} =	8	ft.	
Rail Anchor Slab Thickness	t_{slab} =	12	in.	
Rail Anchor Slab Length	l_{rail} =	30.0	ft.	*(Length between expansion joints)*
Asphalt Overlay Thickness	$t_{asph.}$ =	3	in.	
Bridge Rail Type 7 Width	w_{rail} =	18	in.	
Coping Depth	h_{coping} =	12	in.	
Coping Width	w_{coping} =	8	in.	
Retaining Wall Thickness	t_{wall} =	12	in.	

Rail Anchor Slab Overturning

The overturning moment (M_r) caused by the impact of the vehicle shall be less than the stabilizing moment (M_n) created by the rail anchor slab dead weight. As show in Figure 1, the point of rotation, Point A, is assumed to be at the top, back face of the retaining wall and the structural backfill. In this design example, compressible joint material is placed on top of the wall to protect it, allowing the rail anchor slab to rotate before coming into contact with the wall. The maximum length of rail anchor slab assumed to resist the overturning moment is 60 ft. This limit is assumed to be the extents of rigid body behavior in rail anchor slabs,and is often governed by the spacing of expansion joints perpendicular to the CL of the roadway.

$\phi M_n = \phi \sum(\text{DL Moments}) \geq M_u = \gamma_{CT} M_r$ NCHRP Report 663 (7-3,7-4)

$M_r = L_s H_a$

Test Level		TL-4		CDOT BDM 13.3.3
Resistance Factor	ϕ =	0.9		NCHRP Report 663 A1.4.3
Collision Load Factor	γ_{CT} =	1.0	(Extreme Event II)	AASHTO Table 3.4.1-1
Static Load Equivalent	L_s =	10.0	kip	NCHRP Report 663
Height of Impact Above Roadway	H_e =	32	in.	AASHTO A13.2-1
Dist. from B.F. Rail to 'Pt. A'	l_a =	1.71	ft.	
Dist. from C.G. to 'Pt. A'	l_b =	See table below		
Dist. from Impact Load to 'Pt. A'	h_a =	3.92	ft.	$(h_a = H_e + t_{asph.} + t_{slab})$
Factored Overturning Moment	M_u =	39.2	k-ft.	

To calculate M_n, the dead loads are tabulated and multiplied by the distance from their center of gravity to Point A (l_b). The distance between expansion joints in this example is 30 ft.

Tabulation of Dead Load Moments about Point A

*Weight = Area * $\gamma_{conc.}$*

*Moment = Weight * l_b*

*Total DL Moment = Moment * l_{rail}*

Ref. B-606-7A for rail weight and C.G. from BDM Ex. 6

	Height (ft.)	Width (ft.)	Weight (k/ft.)	l_b (ft.)	Moment (k-ft/ft.)	Total DL Moment (k-ft.)
Type 7 Bridge Rail			0.486	-1.14	-0.55	-16.6
Coping	1.00	0.67	0.10	-1.38	-0.14	-4.1
Slab	1.00	8.00	1.20	2.29	2.75	82.5
Asphalt	0.25	6.50	0.24	3.04	0.72	21.7

M_n = 83.5 k-ft.

M_u = 39.2 k-ft. < ϕM_n = 75.2 k-ft. **OK**

Rail Anchor Slab Sliding

Check the rail anchor slab for resistance to sliding (P_n) along its base. The weight of the slab and rail resists the impact load through friction between the concrete-soil interface. In the absence of project-specific soil information, the coefficient of friction is taken from AASHTO Table 3.11.5.3-1. The soil is assumed to be a silty or clayey fine to medium sand.

$\phi_T P_n = \phi_T W_{DL} \tan\phi_r \geq P_u = \gamma_{CT} L_s$ NCHRP Report 663 (7-1,7-2)

$w_{DL} = \sum(\text{Weight})\ l_{rail}$

Static Load Equivalent	L_s =	10.0	kip	NCHRP Report 663
Total Factored Horizontal Force	P_u =	10.0	kip	
Total Dead Load	W_{DL} =	60.7	kip	
Coefficient of Friction	$\tan\phi_r$ =	0.35		AASHTO Table 3.11.5.3-1
Sliding Resistance	P_n =	21.3	kip	
Sliding Resistance Factor	ϕ_T =	0.8		AASHTO Table 10.5.5.2.2-1
Factored Sliding Resistance	ϕP_n =	17.0	kip	

P_u = 10.0 kip < ϕP_n = 17.0 kip **OK**

Rail Anchor Slab Reinforcing

The critical section of the rail anchor slab shall have sufficient flexural strength (M_n) to resist the impact load and the overhanging dead load of the structural components past Point A. NCHRP Report 663 recommends designing the slab to the appropriate impact loading from AASHTO Table 13.2-1. The structural integrity of the components was not evaluated for use with the static equivalent loads.

$w_{s.o.} = t_{slab}\ l_a\ \gamma_{conc.}$ $e_{s.o.} = l_a / 2$ $M_{slab} = w_{s.o.}\ e_{s.o.}$

(weight due to asphalt is negligible)

Slab Overhang Weight	$w_{s.o.}$ =	0.26	k / ft.
Slab Overhang Eccentricity	$e_{s.o.}$ =	-0.85	ft.
Slab Overhang Moment	M_{slab} =	-0.22	k-ft/ ft.

The impact load is distributed over the length L_C of the barrier rail. L_C is the critical length of the yield line as calculated in Example 6.3 of this BDM. The rail anchor slab reinforcing is placed in the top of the slab and is designed in a 1 ft. strip.

Critical Yield Line Length	L_C =	**10.74**	ft.	AASHTO A13.3.1-2, Ex. 6
Impact Load	F_t =	**54.0**	kip	AASHTO A13.2-1
Dead Load Factor	γ_{DC} =	**1.25**		AASHTO Table 3.4.1-2
Collision Factor	γ_{CT} =	**1.00**		AASHTO Table 3.4.1-1

$M_u = \gamma_{DC}\ (M_{slab}+M_{rail}+M_{coping}) + \gamma_{CT}\ F_t\ H_e / L_C$

(Moment for rail and coping calculated in table)

Factored Moment M_u = 14.54 k-ft./ ft.

Design Section

$\Phi\ Mn = \Phi\ A_s\ f_s\ (d_s - a/2)$ AASHTO 5.6.3.2.2

$a = A_s\ f_y / 0.85\ f'_c\ b$

$c = \beta 1 / a$ AASHTO 5.6.3.2.2

$\beta_1 = 0.85 - 0.05\ (f'_c - 4) \geq 0.65$, for $f'_c > 4$ ksi AASHTO 5.6.2.2

$d = h - C_{TOP} - d_{bar} / 2$

$\varepsilon_s = 0.003\ (d - c) / c$ AASHTO 5.6.2.1

Reinforcement Strength	f_y =	**60**	ksi	
Concrete Strength	f'_c =	**4.5**	ksi	Concrete Class D
Stress Block Factor	β_1 =	0.825		
Strip Width	b =	**12**	in.	
Section Height	h =	12	in.	*(thickness of the slab)*
Top Reinforcing Cover	C_{TOP} =	**2.5**	in.	*(Ref. B-504-V1)*
Resistance Factor	Φ =	**0.9**		*(assume tension controlled)* AASHTO 5.5.4.2.1
Depth to Reinforcing	d =	9.19	in.	*(assume #5 bar)*

Try a Reinforcing Pattern	Try	**# 5**	Bar @	**8**	" C.C.
Diameter of Reinforcing	d_{bar} =	0.625	in.		
Area of Steel per Strip	$A_{s\text{-}prov.}$ =	0.47	in^2		

Net Tensile Strain ε_s = 0.034 > 0.005 **OK**

M_u = 14.54 k-ft/ ft. < $\Phi\ M_n$ = 18.59 k-ft/ ft. **OK**

<u>USE #5 bars @ 8" C.C.</u> *(Typically these bars are also placed in the bottom mat)*

Check Minimum Reinforcement Requirement

AASHTO 5.6.3.3

Flexural Cracking Variability Factor	γ_1 =	**1.6**		
Ratio f_y/f_u	γ_3 =	**0.67**		
Modulus of Rupture	$f_r = 0.24\ (f'_c)^{0.5}$ =	0.51	ksi	AASHTO 5.4.2.6
Section Modulus	$S_c = 1/6\ b\ h^2$ =	288	in^3	

Cracking Moment	$M_{cr} = \gamma_3 \gamma_1 f_r S_c =$	13.1	k-ft./ ft.	***CONTROLS min reinf. & is < Φ Mn***
1.33 * Factored Moment	$1.33\ M_u =$	19.3	k-ft./ ft.	**OK**

Note: Check Crack Control by Distribution of Reinforcement AASHTO 5.6.7
Note: Check Development Length of Flexural Bars AASHTO 5.10.8.2.4
Note: Check Temperature and Shrinkage Steel AASHTO 5.10.6

Rail Anchor Slab Summary