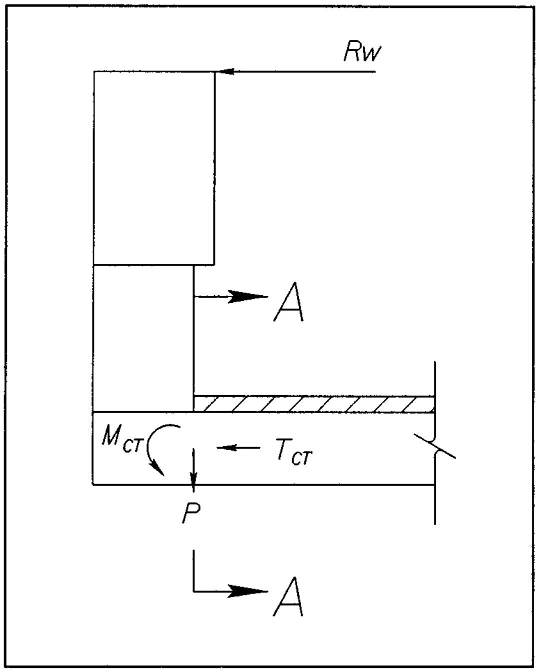
SUMMARY OF CORRAL RAIL COLLISION FORCES FOR DESIGN:



R_w = 260 kN

@ Section A-A:

P = 5.08 kN/m (w/o curb)
= 5.88 kN/m (w/curb)

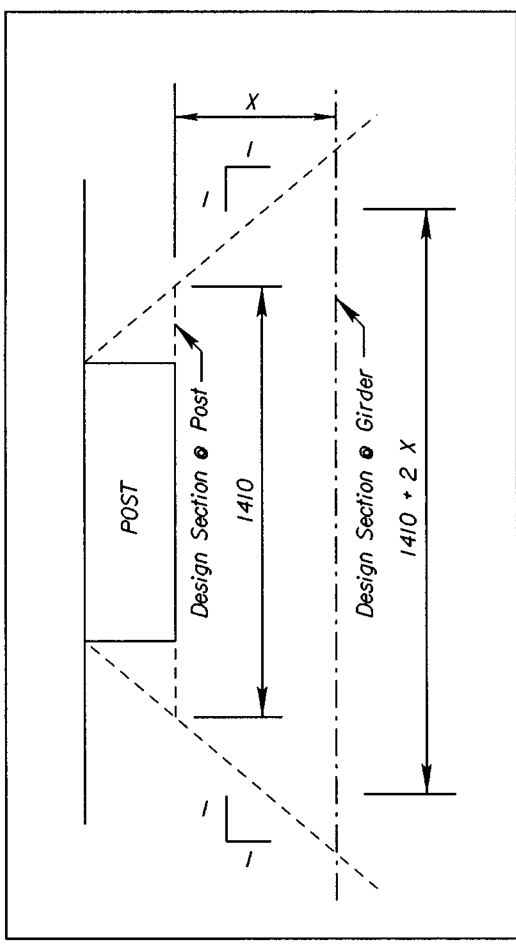
$\mathbf{M_{CT}}$ = 45.5 kN·mm/mm

$\mathbf{T_{CT}}$ = 184 N/mm

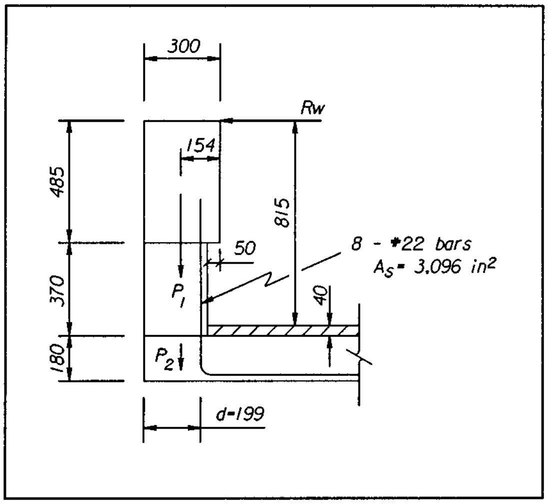
Minimum transverse deck overhang A_s to resist rail hit:

Top slab = 0.800 mm^2/mm

Total, top and bottom of Slab = 1.6 mm^2/mm



Overhang Design for Rail Collision Forces using STAAD Finite Element Model.
(815 mm Corral Rail)



Assumptions:

Fy = 420 MPa
F'c = 30 MPa
Post Width = 910 mm

P_1 = 4.02 kN/m
P_2 = [(180)(250)(2400 kg/m^3) x 9.807 m/s^2]/1 x 10^9
= 1.06 kN/m

Flexural Capacity of Rail:

$$a = \frac{A_s\, f_y}{0.85(f'_c)b} = \frac{3096(420)}{0.85(30)(910)} = 56.04 \text{ mm}$$

$$M_C = \theta\, As\, fy\left(d - \frac{a}{2}\right) = 1.0(3096)(420)\left(199 - \frac{56.04}{2}\right)$$

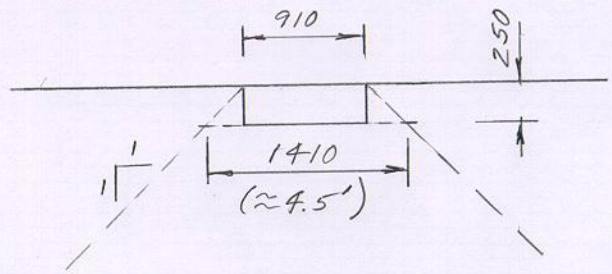
$$= 222{,}331{,}185 \text{ N·m}$$

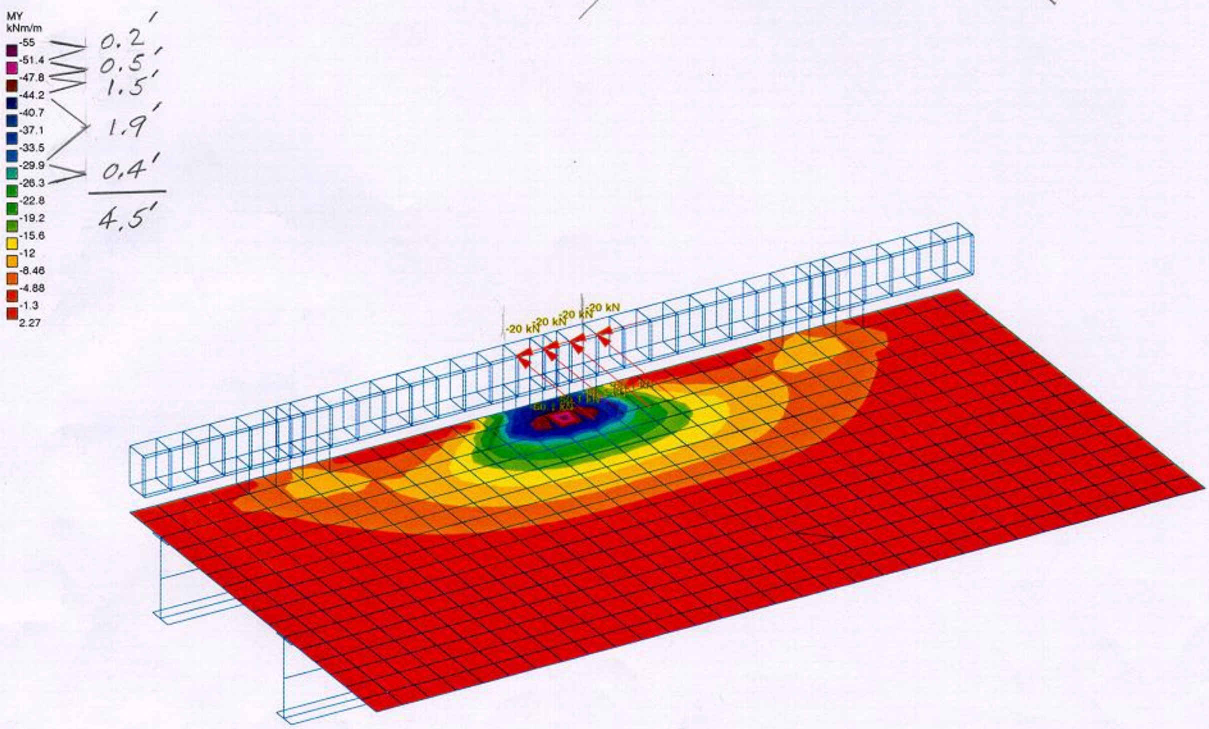
$$= 222.3 \text{ kN·m}$$

$\mathbf{R_w}$ = M_C/y = 222.3/0.855 = **260 kN** > 240 kN required (ok)

A finite element STAAD Model was constructed of a superstructure for a girder bridge with an 815 mm corral rail. The superstructure is a linear elastic, thin plate, four node, quadratic isoparmetric slab model. The model consists of two girders spaced at 2440 mm and four diaphragms spaced at 2440 mm. The deck thickness is 210 mm. The model assumes the deck is partially composite with the girders in the transverse direction.

KDOT Bridge Design (Special Assignments)	Job No	Sheet No 1 (1)	Rev
Software licensed to Kansas Dept of Transportation	Part		
Job Title Rail Hit	Ref		
	By JPJ	Date 14-Aug-00	Chd
Client KDOT	File slab_01.std	Date/Time 30-Jan-2001 11:37	





Moment w/ 240 Kn Transverse and 80 Kn of Longitudinal

Different widths of overhangs were modeled. The shortest overhangs gave the highest stresses, as expected. Several girder types where modeled; the type of girder (welded plate, rolled beam or concrete) did not significantly affect stresses in the overhang.

A force of 240 kN was applied perpendicular to the rail. The resulting stress contours in the slab are displayed on page 2 of 7.

From the STAAD Model, compute the average moment at the base of the Post. Distribute the moment over a distance of the post width plus (2 x post depth): 910 + 2(250) = 1410 mm. Since the STAAD Model consist of one-foot segments, conservatively round down to the nearest 0.5 foot. Therefore, the moment will be distributed over a width of 1372 mm (4.5 feet). (Note: the attached STAAD Model moment contour is for a force of 240 kN):

53.2 kN·m/m	x	61 mm	=	3 245 $kN{\cdot}m^2/m$
49.6 " " "	x	153 "	=	7 589 " "
46.0 " " "	x	457 "	=	21 022 " "
37.1 " " "	x	579 "	=	21 481 " "
28.1 " " "	x	122 "	=	3 428 " "
		1372 mm		56 765 $kN{\cdot}m^2/m$

Average moment @ base of Post due to collision:

$$\frac{56\,765\ kN \cdot m^2/m}{1372\ mm} = 41.4\ \text{kN·mm/mm}$$

Adjust for 260 kN rail hit: $\frac{260}{240}(41.4) = 44.9$ kN·mm/mm

Additional dead load moment at base of rail due to Rail (P_1) and Slab (p_2):

M_{dl} = 4.02 kN/m x 0.104 m + 1.06 kN/m x 0.125 m
= 0.55 kN·m/m

Total moment a face of rail:

$\mathbf{M_{ct}}$ = 44.9 + 0.55 = **45.5 kN·mm/mm**

The above moment is significantly less than that computed assuming the post to be fixed at the base. This is due to the flexibility of the system as a whole that distributes the moments much further than that assumed for a fixed model. The torsional rigidity of the rail also assists in transmitting forces to the adjacent posts.

From the slab design (not included in this paper) have:

#13 & #16 @ 170 mm (alt.) in top of slab: $A_s = 0.964\ mm^2/mm$
#13 @ 170 mm in bottom of slab: $A_s = 0.759\ mm^2/mm$

In addition, at each post, have 5-#16 SP1 bars (See Bridge Base Sheet Standard BR182C SI): $A_s = 995\ mm^2$

Moment capacity of slab at base of post:

$d = 180 - 35 - ½(16) = 137\ mm$

$A_s = 0.964\ mm^2/mm + 995\ mm^2/1410\ mm$
$= 1.70\ mm^2/mm$

$$a = \frac{A_s f_y}{0.85(f'_c)b} = \frac{1.70(420)}{0.85(30)(1)} = 28\ mm$$

$\phi M_u = \phi(A_s)(f_y)(d - a/2) = 1.0(1.70)(420)(137-28/2)/1000$

ϕM_u = 87.8 kN·mm/mm

Reduce moment capacity due to tension stress in slab caused by rail hit:

T_{ct} = 260 kN/1410 mm = **184 N/mm**

Assume the interaction curve between moment and axial force is a straight line:

$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \le 1.0$, therefore: $M_u \le \phi\ M_n\ (1 - P_u/\phi\ P_n)$

$P_u = T_{ct} = 184\ N/mm$

$\phi P_n = \phi A_{sr} f_y$ $\quad A_{sr} =$ 0.964 mm^2/mm (top slab)

0.759 mm^2/mm (btm. Slab)

$\underline{1.411}$ 5-#16 bars $\left(\frac{199(5)2}{1410}\right)$

3.134 mm^2/mm

$$\phi A_{sr} f_y = 1.0(3.134)(420) = 1316 \text{ N/mm}$$

$$\mathbf{M_u} \le 1.0(87.8)\left(1-\frac{184}{1316}\right) \mathbf{= 75.5\ kN{\cdot}mm/mm > 45.5\ kN{\cdot}mm/mm\ (OK)}$$

<u>Check development length of # 16 slab bar at post</u>:

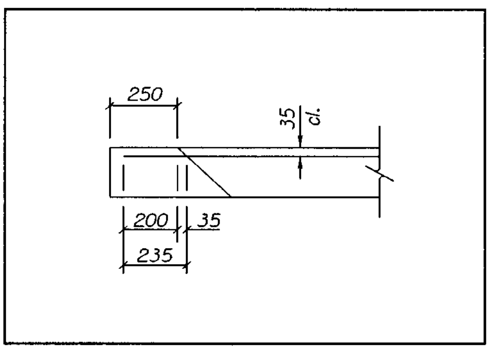
$$\frac{0.02 A_s F_y}{\sqrt{f'_c}} = \frac{0.02(199)(420)}{\sqrt{30}} = 305.2 \ge 0.06 d_b f_y$$

$$\ge 0.06(15.9)(420)$$

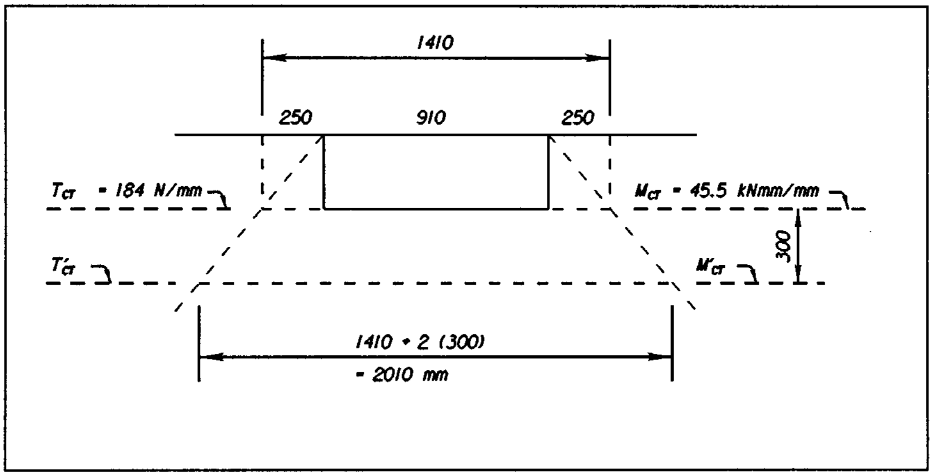
$$\ge 400.7 \text{ (controls)}$$

Modify by epoxy steel: 1.2
spacing: 0.8
req'd/actual: 45.5/75.5

Development Length = 400.7 x 1.2 x 0.8 x 45.5/75.5
= 232 mm < 235 mm (OK)



Check slab capacity at end of SP1 bars (300 mm from post):



$$M'_{ct} = \frac{1410}{2010}(45.5) = 31.9 \text{ kN·mm/mm}$$

$$T'_{ct} = \frac{1410}{2010}(184) = 129.1 \text{ N/mm}$$

$A_{s(top)}$ = 0.964 mm^2/mm

$A_{s(btm)}$ = <u>0.759 mm^2/mm</u>

1.723 mm^2/mm total A_s

Slab capacity: $a = \frac{0.964(420)}{0.85(30)(1)} = 15.88$ mm

M_u = 1.0(0.964)(420)(137-15.88/2)/1000

= 52.25 kN·mm/mm

Tension reduction:

$$M_n = 52.25\left(1 - \frac{129.1}{1.723(420)}\right) = 42.9 \text{ kN·mm/mm} > 31.9 \text{ kN·mm/mm (OK)}$$