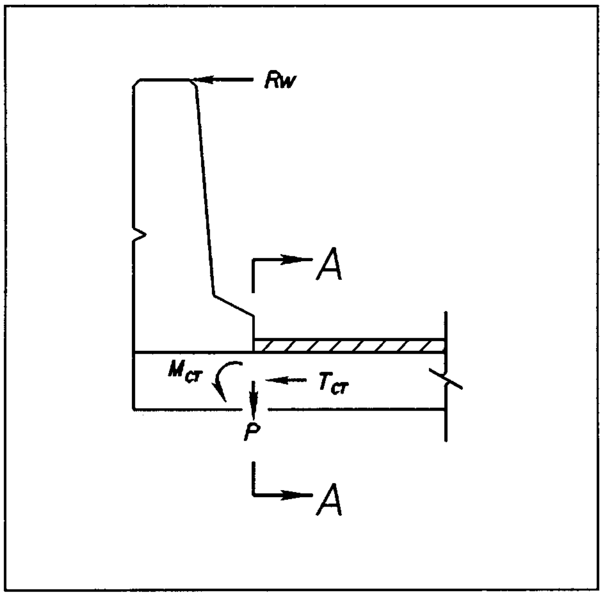
**SUMMARY OF BARRIER RAIL COLLISION FORCES FOR DESIGN:**




$R_w$ = 338 Kn (See Sheet 5)

$L_c$ = 2169 mm (See Sheet 5)

@ Section A-A:

**P** = 6.83 N/mm

$\mathbf{T_{CT}}$ = 87.1 N/mm

$\mathbf{M_{CT}}$ = 37.8 kN·mm/mm

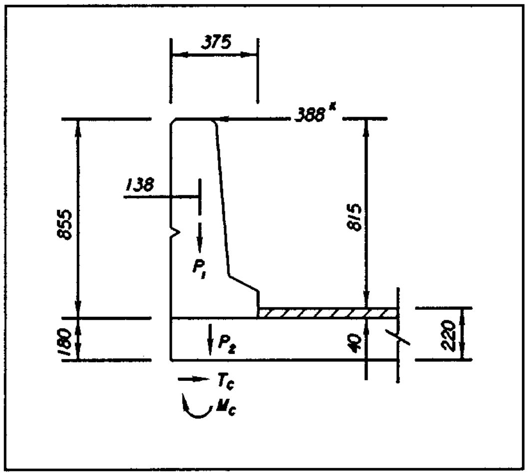
Minimum transverse deck overhang $A_s$ to resist rail hit:

Top Slab: 0.800 $mm^2$/mm

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

**Overhang Design for Barrier Collision Forces using STAAD Finite Element Model**

(815 mm Barrier Rail)




Assumptions:

$F_y$ = 420 MPa
$F'_c$ = 30 MPa

$P_1$ = 5.24 kN/m
$P_2$ = [(0.375)(.180)(2400 kg/m$^3$) X 9.807 m/s$^2$]/ 1 x $10^9$
= 1.59 kN/m

Ref: Bridge Standard BR184A SI

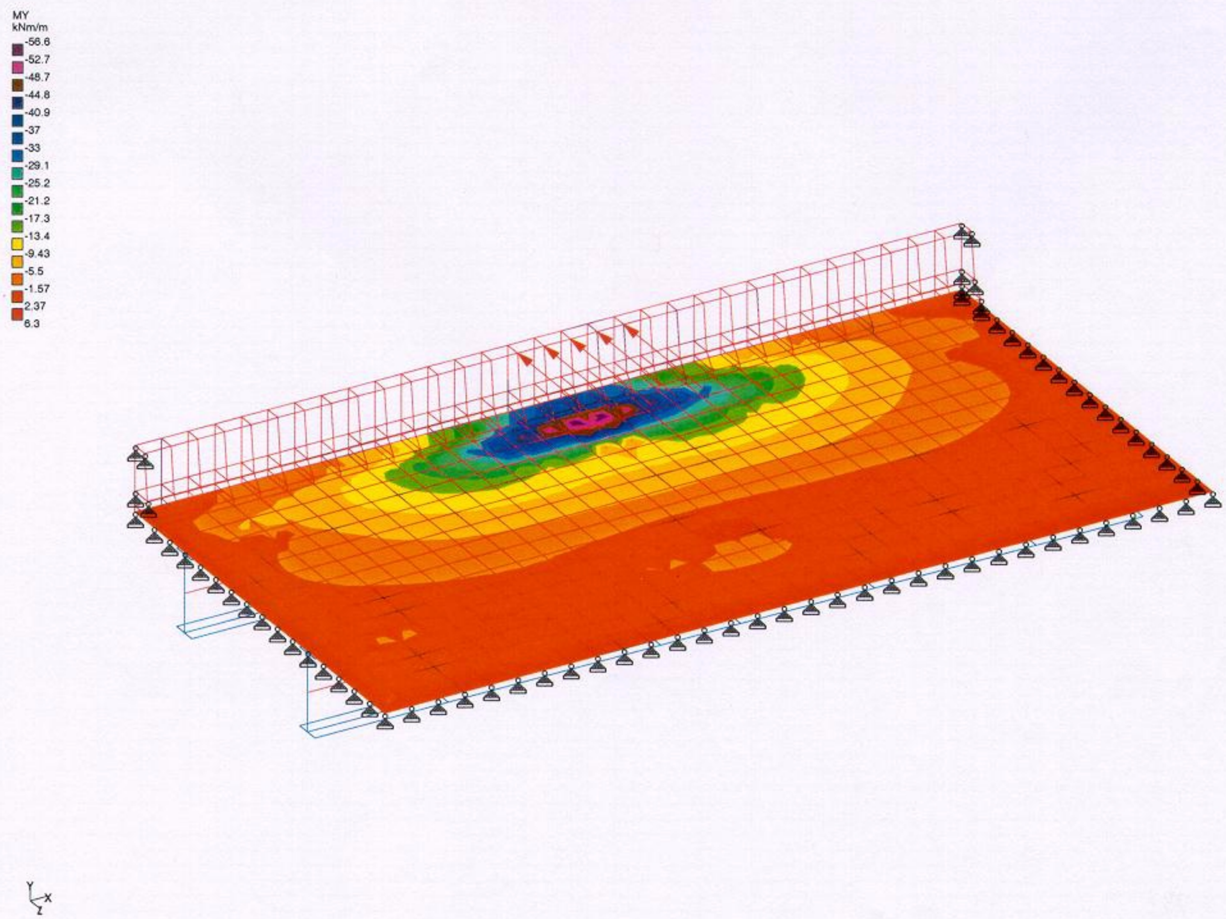
From yield line theory analysis of rail hit, (not included in this paper), $R_w$ = 338 kN and $L_c$ = 2169 mm (see page 5 of 6 for assumptions and sketch).

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.

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

A force of 338 kN was applied to the STAAD Finite Element Model. The resulting stress contours in the slab are displayed on page 2 of 6.

The average moment at the base of rail is computed over a distance of 3880 mm [2169 mm + 2(855 mm)]. Since the STAAD Model consist of one-foot (305 mm) segments, conservatively




*815 mm F-2 Barrier w/76k hit (Moment in Slab)* round down to the nearest whole foot. Thus, the average moment is computed at the base of the rail over a distance of 3658 mm (12 feet).

| | | | | | | |
|---|---|---|---|---|---|---|
| 54.65 | kN·mm/mm | x | 153 mm | = | 8 361 | $kN \cdot mm^2/mm$ |
| 50.70 | " " " | x | 762 " | = | 38 633 | " " |
| 46.75 | " " " | x | 457 " | = | 21 365 | " " |
| 35.00 | " " " | x | 914 " | = | 31 990 | " " |
| 27.15 | " " " | x | 457 " | = | 12 408 | " " |
| 23.20 | " " " | x | 762 " | = | 17 678 | " " |
| 15.35 | " " " | x | 153 " | = | 2 349 | " " |
| | | | 3658 mm | = | 132 784 | $kN \cdot mm^2/mm$ |

Average moment @ base of Barrier due to collision:

$$\frac{132\,784\ kN \cdot mm^2 / mm}{3658\ mm} = 36.3\ kN \cdot mm / mm$$

Additional dead load moment at the base of the Barrier due to Barrier force ($P_1$) and Slab force ($P_2$):

$M_{dl}$ = 5.24 kN/m x 0.237 m + 1.59 kN/m x 0.375/2

= 1.54 kN·mm/mm

$\mathbf{M_{CT}}$ = 1.54 + 36.3 = **37.8 kN·mm/mm**

The above moment is significantly less than that computed assuming the barrier 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.

From the slab design (not included) have:

#13 & #16 bars @ 170 mm (top of slab): $A_s$ = 0.964 $mm^2/mm$

#13 bars @ 170 mm in bottom of slab: $A_s$ = 0.759 $mm^2/mm$

**Moment Capacity of slab at base of barrier**:

$d = 180 - 35 - ½(16) = 137$ mm

$A_s = 0.964$ mm$^2$/mm

$$a = \frac{A_s f_y}{0.85(f'_c)\,b} = \frac{0.964(420)}{0.85(30)(1)} = 15.88\ mm$$

$\boldsymbol{\phi M_n} = \phi(A_s)(F_y)(d - a/2) = 1.0[(0.964)(420)(137-15.88/2)]$

$=$ **52.25 kN·mm/mm**

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

$\mathbf{T_{CT}}$ = 337.8 kN / (2169 + 2(855)) = **87.1 N/mm**

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

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

$\phi M_n = 52.25$ kN·mm/mm

$P_u = T_{CT} = 87.1$ N/mm

$\phi P_n = \phi A_{sr} F_y$

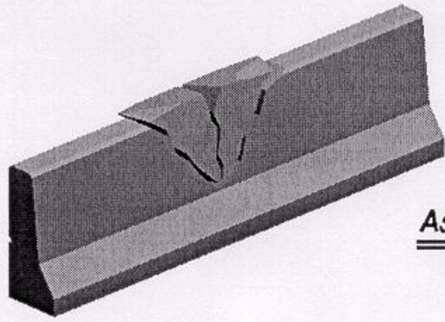
Asr = 0.964 mm$^2$/mm (top of slab)

0.759 mm$^2$/mm (btm. of slab)

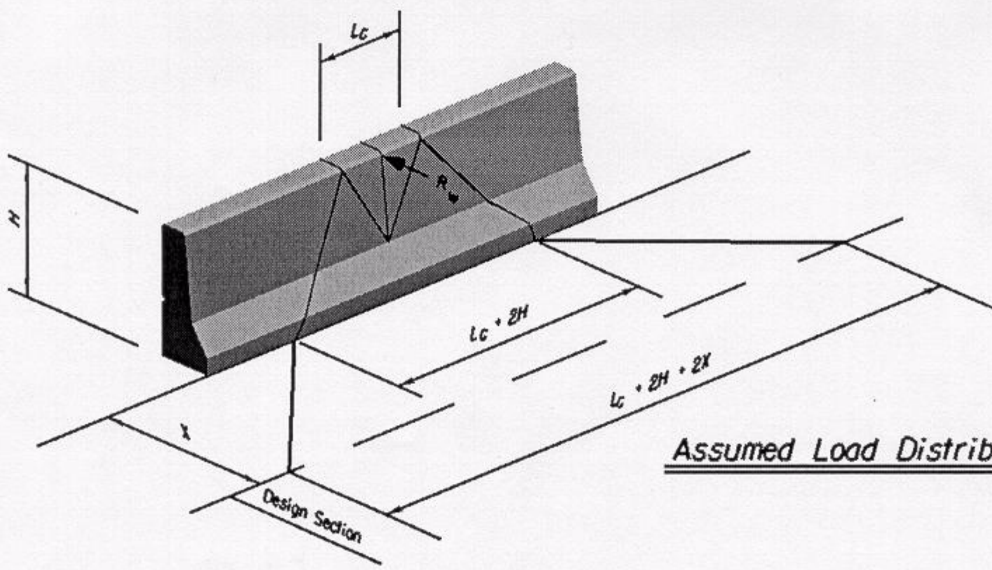
Total = 1.723 mm2/mm

$\phi A_{sr} F_y = 1.0(1.723)(420) = 723.7$ N/mm

$\mathbf{M_u} \leq 1.0(52.25)\left(1 - \frac{87.1}{723.7}\right)$ = **46.0 kN·mm/mm > 37.8 kN·mm/mm (OK)**



Assumed Failure Mode




Assumed Load Distribution