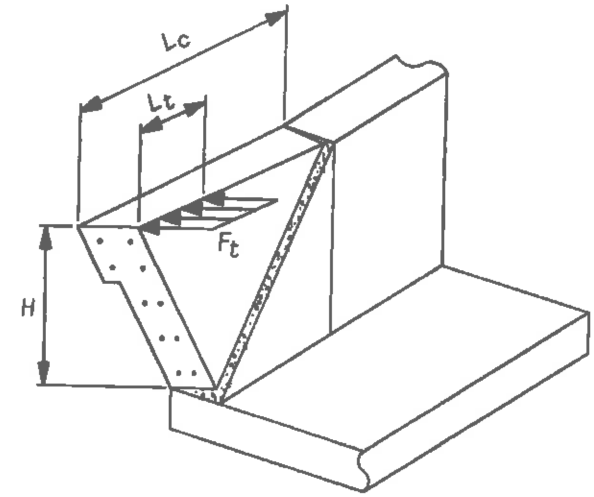

Figure CA13.3.1-2—Yield Line Analysis of Concrete Parapet Walls for Impact near End of Wall Segment

A13.3.2—Post-and-Beam Railings

Inelastic analysis shall be used for design of post-and-beam railings under failure conditions. The critical rail nominal resistance, *R*, when the failure does not involve the end post of a segment, shall be taken as the least value determined from Eqs. A13.3.2-1 and A13.3.2-2 for various numbers of railing spans, *N*.

CA13.3.2

A basis for applying inelastic analysis is shown in Figure CA13.3.2-1.

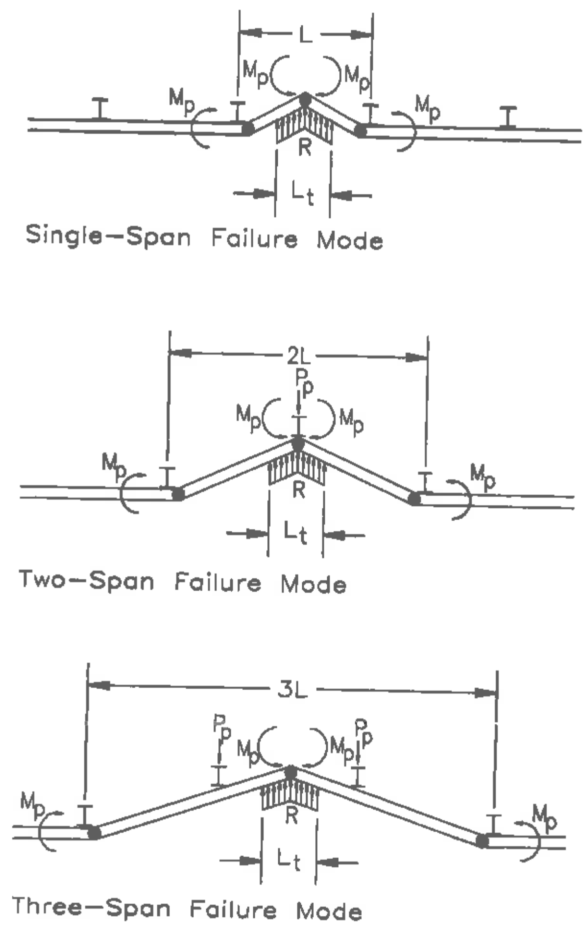

Figure CA13.3.2-1—Possible Failure Modes for Post-and-Beam Railings

- For failure modes involving an odd number of railing spans, N:

$$R = \frac{16M_p + (N-1)(N+1)P_p L}{2NL - L_t} \quad \text{(A13.3.2-1)}$$

- For failure modes involving an even number of railing spans, N:

$$R = \frac{16M_p + N^2 P_p L}{2NL - L_t} \quad \text{(A13.3.2-2)}$$

where:

L = post spacing or single-span (ft)

M_p = inelastic or yield line resistance of all of the rails contributing to a plastic hinge (kip-ft)

M_{post} = plastic moment resistance of a single post (kip-ft)

P_p = shear force on a single post which corresponds to M_{post} and is located $\bar{Y}$ above the deck (kips)

R = total ultimate resistance, i.e., nominal resistance, of the railing (kips)

L_t, L_L = transverse length of distributed vehicle impact loads, F_t and F_L (ft)

For impact at the end of rail segments that causes the end post to fail, the critical rail nominal resistance, R, shall be calculated using Eq. A13.3.2-3.

- For any number of railing spans, N.

$$R = \frac{2M_p + 2P_p L\left(\sum_{i=1}^{N} i\right)}{2NL - L_t} \quad \text{(A13.3.2-3)}$$

This design procedure is applicable to concrete and metal post and beam railings.

The post on each end of the plastic mechanism must be able to resist the rail or beam shear.

For multiple rail systems, each of the rails may contribute to the yield mechanism shown schematically in Figure CA13.3.2-1, depending on the rotation corresponding to its vertical position.

A13.3.3—Concrete Parapet and Metal Rail

The resistance of each component of a combination bridge rail shall be determined as specified in Articles A13.3.1 and A13.3.2. The flexural strength of the rail shall be determined over one span, R_R, and over two spans, R'_R. The resistance of the post on top of the wall, P_p, including the resistance of the anchor bolts or post shall be determined.

The resistance of the combination parapet and rail shall be taken as the lesser of the resistances determined for the two failure modes shown in Figures A13.3.3-1 and A13.3.3-2.

CA13.3.3

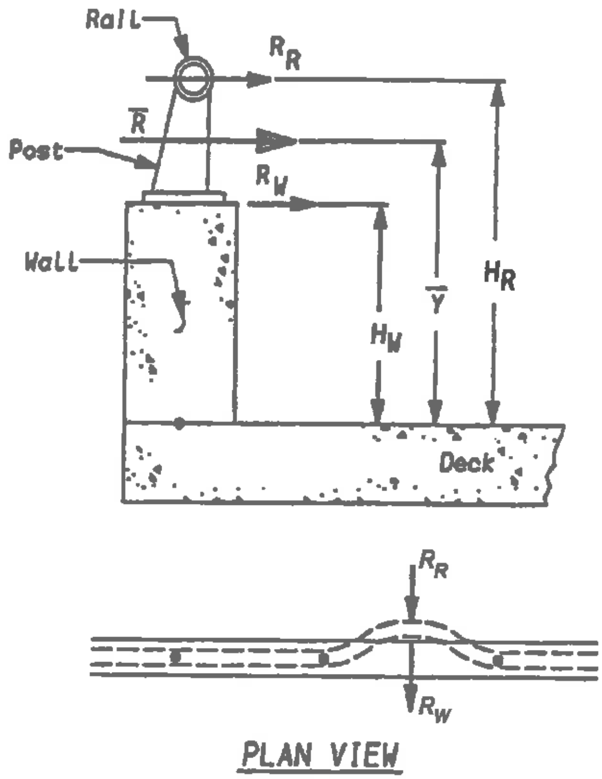

Figure A13.3.3-1—Concrete Wall and Metal Rail Evaluation—Impact at Midspan of Rail

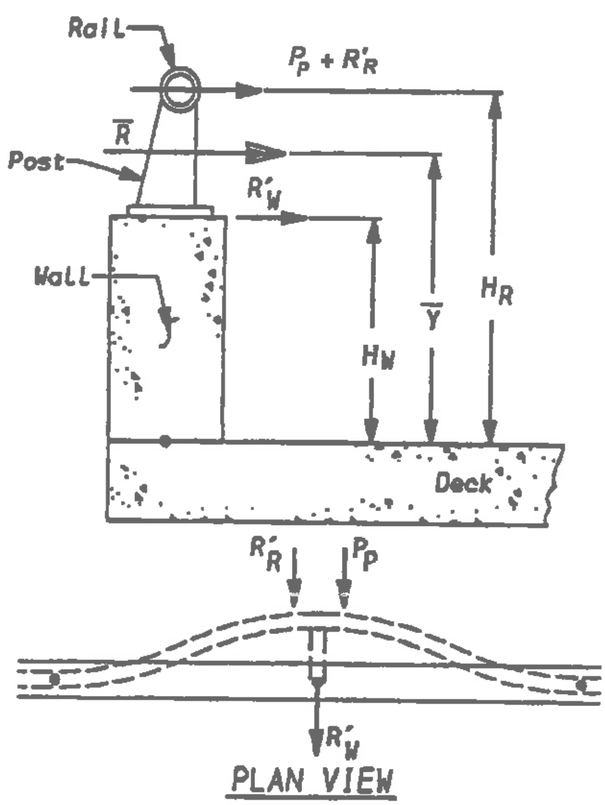

Figure A13.3.3-2—Concrete Wall and Metal Rail Evaluation—Impact at Post

Where the vehicle impact is at midspan of the metal rail, as illustrated in Figure A13.3.3-1, the flexural resistance of the rail, R_R, and the maximum strength of the concrete wall, R_w, shall be added together to determine the combined resultant strength, $\overline{R}$, and the effective height, $\overline{Y}$, taken as:

$$\overline{R} = R_R + R_w \qquad \text{(A13.3.3-1)}$$

$$\overline{Y} = \frac{R_R H_R + R_w H_w}{\overline{R}} \qquad \text{(A13.3.3-2)}$$

where:

R_R = ultimate capacity of rail over one span (kips)
R_w = ultimate capacity of wall as specified in Article A13.3.1 (kips)
H_w = height of wall (ft)
H_R = height of rail (ft)

The commentary to Article CA13.2 applies.

Where the vehicle impact is at a post, as illustrated in Figure A13.3.3-2, the maximum resultant strength, $\overline{R}$, shall be taken as the sum of the post capacity, P_p, the rail strength, R'_R, and a reduced wall strength, R'_w, located at a height $\overline{Y}$.

$$\overline{R} = P_p + R'_R + R'_w \qquad \text{(A13.3.3-3)}$$

$$\overline{Y} = \frac{P_p H_R + R'_R H_R + R'_w H_w}{\overline{R}} \qquad \text{(A13.3.3-4)}$$

in which:

$$R'_w = \frac{R_w H_w - P_p H_R}{H_w} \qquad \text{(A13.3.3-5)}$$

where:

P_p = ultimate transverse resistance of post (kips)
R'_R = ultimate transverse resistance of rail over two spans (kips)
R'_w = capacity of wall, reduced to resist post load (kips)
R_w = ultimate transverse resistance of wall as specified in Article A13.3.1 (kips)

It should also be recognized that a maximum effective height, $\overline{Y}$, equal to the centroid rail height, H_R, could be obtained, but at a reduced resultant strength, $\overline{R}$, equal to the post capacity, P_p, and rail capacity, R'_R, only.

The analysis herein does not consider impacts near open joints in the concrete wall or parapet. The metal rail will help distribute load across such joints. Improved rail resistance will be obtained if the use of expansion and contraction joints is minimized.

For impact near the end of railing segments, the nominal resistance may be calculated as the sum of the wall resistance, calculated using Eq. A13.3.1-3, and the metal rail resistance over one span, calculated using Eq. A13.3.2-3.

A13.3.4—Wood Barriers

Wood barriers shall be designed by elastic linear analysis with member sections proportioned on the basis of their resistances, specified in Section 8, using the strength limit states and the applicable load combinations specified in Table 3.4.1-1.

CA13.3.4

A limit or failure mechanism is not recommended for wood railings.

A13.4—DECK OVERHANG DESIGN

A13.4.1—Design Cases

Bridge deck overhangs shall be designed for the following design cases considered separately:

Design Case 1: the transverse and longitudinal forces specified in Article A13.2 Extreme Event Load Combination II limit state

Design Case 2: the vertical forces specified in Article A13.2—Extreme Event Load Combination II limit state

Design Case 3: the loads, specified in Article 3.6.1, that occupy the overhang—Load Combination Strength I limit state

For Design Cases 1 and 2, the load factor for dead load, γ_p, shall be taken as 1.0.

The total factored force effect shall be taken as:

$$Q = \Sigma \eta_i \gamma_i Q_i \quad \text{(A13.4.1-1)}$$

where:

η_i = load modifier specified in Article 1.3.2
γ_i = load factors specified in Tables 3.4.1-1 and 3.4.1-2, unless specified elsewhere
Q_i = force effects from loads specified herein

A13.4.2—Decks Supporting Concrete Parapet Railings

For Design Case 1, the deck overhang may be designed to provide a flexural resistance, M_s, in kip-ft/ft which, acting coincident with the tensile force T in kip/ft, specified herein, exceeds M_c of the parapet at its base. The axial tensile force, T, may be taken as:

$$T = \frac{R_w}{L_c + 2H} \quad \text{(A13.4.2-1)}$$

where:

R_w = parapet resistance specified in Article A13.3.1 (kips)
L_c = critical length of yield line failure pattern (ft)
H = height of wall (ft)
T = tensile force per unit of deck length (kip/ft)

Design of the deck overhang for the vertical forces specified in Design Case 2 shall be based on the overhanging portion of the deck.

CA13.4.2

If the deck overhang capacity is less than that specified, the yield line failure mechanism for the parapet may not develop as shown in Figure CA13.3.1-1, and Eqs. A13.3.1-1 and A13.3.1-2 will not be correct.

The crash testing program is oriented toward survival, not necessarily the identification of the ultimate strength of the railing system. This could produce a railing system that is significantly overdesigned, leading to the possibility that the deck overhang is also overdesigned.

A13.4.3—Decks Supporting Post-and-Beam Railings

A13.4.3.1—Overhang Design

For Design Case 1, the moment in kip-ft/ft, M_d, and tensile force, in kip/ft of deck, T, may be taken as:

$$M_d = \frac{12M_{post}}{W_b + d_b} \qquad \text{(A13.4.3.1-1)}$$

$$T = \frac{12P_p}{W_b + d_b} \qquad \text{(A13.4.3.1-2)}$$

For Design Case 2, the punching shear force and overhang moment may be taken as:

$$P_v = \frac{F_v L}{L_v} \qquad \text{(A13.4.3.1-3)}$$

$$M_d = \frac{P_v X}{b} \qquad \text{(A13.4.3.1-4)}$$

in which:

$$b = 2X + \frac{W_b}{12} \leq L \qquad \text{(A13.4.3.1-5)}$$

where:

M_{post} = plastic moment resistance of a single post (kip-ft)

P_p = shear force on a single post which corresponds to M_{post} and is located $\bar{Y}$ above the deck (kips)

X = distance from the outside edge of the post base plate to the section under investigation, as specified in Figure A13.4.3.1-1 (ft)

W_b = width of base plate (in.)

T = tensile force in deck (kip/ft)

d_b = distance from the outer edge of the base plate to the innermost row of bolts, as shown in Figure A13.4.3.1-1 (in.)

L = post spacing (ft)

L_v = longitudinal distribution of vertical force F_v on top of railing (ft)

F_v = vertical force of vehicle laying on top of rail after impact forces F_t and F_L are over (kips)

b = length of deck resisting post strength or shear load

CA13.4.3.1

Vehicle collision on the beam and post railing systems, such as a metal system with wide flange or tubular posts, imposes large concentrated forces and moments on the deck at the point where the post is attached to the deck.

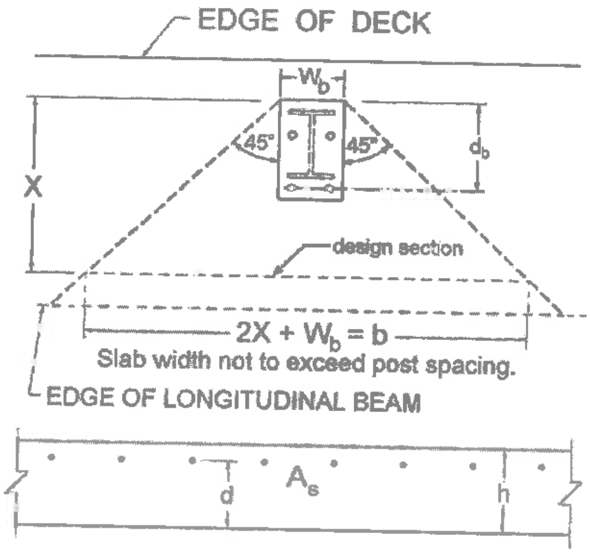

Figure A13.4.3.1-1—Effective Length of Cantilever for Carrying Concentrated Post Loads, Transverse or Vertical

Previous editions of the Standard Specifications distributed railing or post loads to the slab using similar simplified analysis, e.g., "The effective length of slab resisting post loadings shall be equal to $E = 0.8x + 3.75$ ft where no parapet is used and equal to $E = 0.8x + 5.0$ ft where a parapet is used, where x is the distance in ft from the center of the post to the point under investigation."

A13.4.3.2—Resistance to Punching Shear

For Design Case 1, the factored shear may be taken as:

$$V_u = A_f F_y \qquad \text{(A13.4.3.2-1)}$$

The factored resistance of deck overhangs to punching shear may be taken as:

$$V_r = \phi V_n \qquad \text{(A13.4.3.2-2)}$$

$$V_n = v_c \left[W_b + h + 2\left(E + \frac{B}{2} + \frac{h}{2} \right) \right] h \qquad \text{(A13.4.3.2-3)}$$

$$v_c = \left(0.0633 + \frac{0.1265}{\beta_c} \right) \sqrt{f'_c} \leq 0.1265\sqrt{f'_c} \qquad \text{(A13.4.3.2-4)}$$

$$\frac{B}{2} + \frac{h}{2} \leq B \qquad \text{(A13.4.3.2-5)}$$

in which:

$$\beta_c = W_b / d_b \qquad \text{(A13.4.3.2-6)}$$

where:

V_u = factored shear force at section (kips)
A_f = area of post compression flange (in.^{2})
F_y = yield strength of post compression flange (ksi)
V_r = factored shear resistance (kips)
V_n = nominal shear resistance of the section considered (kips)
v_c = nominal shear resistance provided by tensile stresses in the concrete (ksi)
W_b = width of base plate (in.)

CA13.4.3.2

Concrete slabs or decks frequently fail in punching shear resulting from the force in the compression flange of the post, C. Adequate thickness, h, edge distance, E, or base plate size (W_b or B or thickness) should be provided to resist this type failure.

h = depth of slab (in.)
E = distance from edge of slab to centroid of compressive stress resultant in post (in.)
B = distance between centroids of tensile and compressive stress resultants in post (in.)
β_c = ratio of the long side to the short side of the concentrated load or reaction area
f'_c = 28-day compressive strength of concrete (ksi)
φ = resistance factor = 1.0
d_b = distance from the outer edge of the base plate to the innermost row of bolts (in.)

The assumed distribution of forces for punching shear shall be as shown in Figure A13.4.3.2-1.

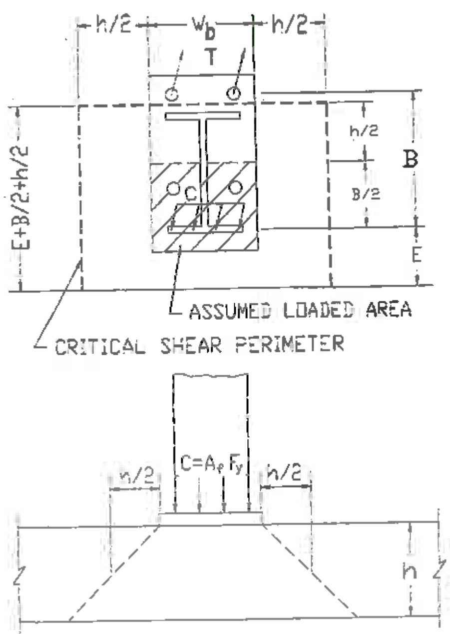

Figure A13.4.3.2-1—Punching Shear Failure Mode

Test results and in-service experience have shown that where deck failures have occurred, the failure mode has been a punching shear-type failure with loss of structural integrity between the concrete and reinforcing steel. Use of various types of shear reinforcement may increase the ultimate strength of the postdeck connection but is ineffective in reducing shear, diagonal tension, or cracking in the deck. Shear resistance can be increased by increasing the slab thickness, base plate width and depth, or edge distance.