GUIDE SPECIFICATIONS FOR BRIDGE RAILINGS

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Appendix B — X

PREFACE

guide specifications have been developed to provide an alternative ... ridge railing design procedures contained in the fourteenth edition of the *AASHTO Standard Specifications for Highway Bridges*. Because it is likely that these guide specifications will become the basis for revisions to the standard specifications, article numbers corresponding to comparable articles in the standard specifications have been used in the guide specifications, except that a letter "G" prefix has been added.

Two concepts underlie these guide specifications. The first is that bridge railing designs should be crash tested to confirm that they will meet the requirements of a specified railing performance level. The second is that bridge railing performance needs differ greatly from site to site over our highway network and that railing designs, and costs, should match site needs (the multiple performance levels concept).

Three bridge railing performance levels and associated crash tests and performance requirements are given in these guide specifications, along with guidance for determining the appropriate railing performance level for a given bridge site. Appendix A to the guide specifications provides guidelines for the structural and geometric design of railings that are likely to meet desired performance level crash test requirements. (Appendix A is intended for guidance only. Provisions therein are not requirements to be imposed beyond the requirements of the guide specifications.) Appendix B discusses the theory, data, and assumptions upon which the guide specifications are based, along with the potential impact of the specifications.

These guide specifications are applicable to railings for new bridges and for bridges being rehabilitated to the extent that railing replacement is obviously appropriate. They are *not* applicable to determining the adequacy of existing railings, when existing railings should be strengthened or replaced, or the method or level of strengthening appropriate for upgrading a substandard existing railing. Such determinations require special study, the outcome of which will depend on site specific factors such as the condition of the existing railing, its performance record (at the site and elsewhere), traffic volume and mix, costs to effect various levels of upgrade, expected time to major rehabilitation or replacement of the bridge, etc. In general, because construction and maintenance costs of the do-nothing option are usually very low or zero, it will be rare that replacement or upgrading of an in-place railing that does not have a recognized poor performance record under its site specific conditions will be justified. Or stated another way, it may be appropriate to do nothing with an effective existing railing with a performance level significantly below that which would be indicated by these guide specifications because the cost to improve the railing would not be justified by the improvement in safety achievable.

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COMMENTARIES ON GUIDE SPECIFICATIONS FOR BRIDGE RAILINGS

SPECIFICATIONS

G2.1 GENERAL

G2.1.1* Notations

- A = Distance from front of vehicle to its center of gravity, ft. (Table G2.7.1.3A)
- A_f = Area of Flange, in^2 (Article G3.7.4.3)
- B = Width of vehicle, ft. (Table G2.7.1.3A)
- b = Flange width, in. (Article G2.7.4.3)
- D = clear unsupported distance between flange components, in. (Article G2.7.4.3)
- d = depth of W or I section, in. (Article G2.7.4.3)
- F_a = allowable axial stress, psi (Article G2.7.4.3)
- F_b = allowable bending stress, psi (Article G2.7.4.2)
- F_v = allowable shear stress, psi (Article G2.7.4.2)
- F_y = minimum yield stress, psi (Article G2.7.4.2)
- f_a = axial compression stress, psi (Article G2.7.4.3)
- H_{cg} = Height of vehicle center of gravity, in. (Table G2.7.1.3A)
- K_c = Traffic Adjustment Factor for Curvature (Article G2.7.1.3, Figure G2.7.1.3A, and Table G2.7.1.3B)
- K_g = Traffic Adjustment Factor for Grade, (Article G2.7.1.3, Figure G2.7.1.3A, and Table G2.7.1.3B)
- K_s = Traffic Adjustment Factor for deck height and under-structure conditions (Article G2.7.1.3, Figure G2.7.1.3B, and Table G2.7.1.3B)
- L = post spacing (Figure G2.7.4)
- R = Ratio of weight assumed to be acting on tractor unit to total vehicle weight (Table G2.7.1.3A)
- t = web thickness, in. (Article G2.7.4.3)
- V = Impact speed, mph (Table G2.7.1.3A)
- V_p = Speed of vehicle when it becomes parallel to railing, mph (Table G2.7.1.3A)
- W = Gross weight of vehicle, Kips (Table G2.7.1.3A)
- w = pedestrian or bicycle loading (Articles G2.7.2.2, G2.7.3.2, and Figure G2.7.4)
- θ = Impact angle, deg. (Table G2.7.1.3A)
- μ = Effective coefficient of friction between railing and impacting vehicle (Table G2.7.1.3A)

G2.2.5 Curbs and Sidewalks

The face of the curb is defined as the vertical or sloping surface on the roadway side of the curb. Horizontal measurements of roadway curbs are from the bottom of the face or, in the case of stepped back curbs, from the bottom of the lower face. A sidewalk or a brush curb located on the highway traffic side of a bridge railing shall be considered an integral part of the railing and shall be subject to the crash test requirements of Article G2.7.1.1.3. The width of a brush curb shall not exceed 9 inches, desirably, should not exceed 6 inches. When curb and gutter sections are used on the roadway approach, at either or both ends of the bridge, the curb height on the bridge shall preferably equal, but may exceed, the curb height on the roadway approach. Changes in curb height shall be uniformly transitioned over a distance equal to or greater than 20 times the change in height. Where no curbs are used on the roadway approaches, the height of the bridge curb above the roadway shall be not less than 6 inches, and preferably not more than 8 inches.

Raised sidewalks on bridges usually should not be used where the approach roadway is not curbed. However, when staged construction, a change in roadway cross section from one end of the bridge to the other, or some other condition requires a raised sidewalk on a bridge with no connecting approach curb, a transition section of sidewalk with a length at least 20 times the height of the sidewalk curb on the bridge shall be provided to ramp the bridge sidewalk to the level of the approach surface.

For recommendations on sidewalk widths see AASHTO *A Policy on Geometric Design of Highways and Streets*.

Where sidewalks are used for pedestrian traffic on urban expressways they shall be separated from the bridge roadway by the use of a traffic railing or combination railing as discussed in Article G2.7.

In those cases where a New Jersey type parapet or other railing or a curb is constructed on a bridge, particularly in urban areas that have curbs and gutters leading to a bridge, the same width between curbs on the approach roadway will be maintained across the bridge structure. A parapet or other railing installed at or near the curb line shall have its ends properly flared, sloped, or shielded.

G2.7 RAILINGS

Railings shall be provided along the edges of structures for protection of traffic and pedestrians. A pedestrian walkway *may* be separated from an adjacent roadway by a traffic railing or combination railing, with a pedestrian railing along the edge of the

* See preface for explanation of article numbering.

structure, except on urban expressways where a pedestrian walkway, if provided, *shall* be separated from the adjacent roadway by a traffic railing or combination railing.

G2.7.1 Traffic Railings and Combination Railings

G2.7.1.1 General

G2.7.1.1.1 Although the primary purpose of traffic railings is to contain vehicles using the structure, consideration should also be given to (a) protection of the occupants of a vehicle in collision with the railing, (b) protection of other vehicles near the collision, (c) protection of persons and property on roadways or other areas underneath the structure, (d) railing cost-effectiveness, and (e) appearance and freedom of view from passing vehicles.

G2.7.1.1.2 The approach end of a parapet or railing shall have an appropriate crashworthy configuration or be shielded by a crashworthy traffic barrier. Traffic barriers on bridge approaches must be properly transitioned to traffic railings on bridges. Bridge-end drainage control should be an integral part of the barrier transition design.

G2.7.1.1.3 To ensure safe performance, traffic railings, combination railings (traffic railings combined with pedestrian railings or bicycle railings), and barrier transitions shall be crash tested and evaluated in accordance with the crash test procedures given in the National Cooperative Highway Research Program Report 230, *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances,* except as otherwise directed in Article G2.7.1.3 of these specifications. In addition, combination railings are to meet the loading requirements for bicycle railings given in Article G2.7.2.2 or for pedestrian railings given in Article G2.7.3.2, as appropriate.

A combination railing may be crash tested and certified for use with a raised sidewalk having unique dimensions. However, a combination railing crash tested with a flush roadway approach surface and with a sidewalk conforming to the dimensions given in Figure G2.7.1.1.3 may be considered as acceptable for use with sidewalks having widths 3.5 feet or greater and heights up to 8 inches, provided the crash test results meet the requirements given in Table G2.7.1.3A under "Crash Test Evaluation Criteria."

G2.7.1.1.4 Variations in traffic volume, speed, vehicle mix, roadway alignment, under-structure activities and conditions, and other factors combine to produce a vast variation in traffic railing performance needs from one site to another. The performance requirements for traffic railings and the criteria for their selection are given in Article G2.7.1.3.

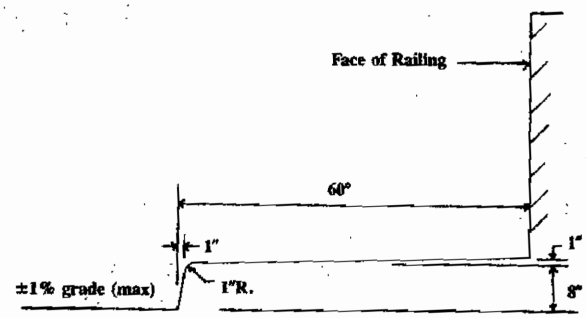


FIGURE G2.7.1.1.3 Standard Raised Sidewalk for Use in Combination Railing Testing.

G2.7.1.2 Geometry

G2.7.1.2.1 Acceptability of traffic railing and combination railing geometry shall be verified through crash testing. However, the minimum height of a traffic railing, measured at its roadway face, from the top of the roadway or from the top of an anticipated future overlay shall not be less than 27 inches.

G2.7.1.2.2 When a traffic railing is located between the roadway and a sidewalk or bikeway, the minimum height of the railing above the surface of the sidewalk or bikeway shall be 24 inches and the railing should have a smooth surface to avoid snag points for pedestrians or cyclists. When a greater height of railing above a sidewalk or bikeway surface is desired to improve comfort or safety of pedestrians or cyclists with a potential of falling over the railings and onto the roadway, the railing may be a traffic railing or a modified combination railing giving a selected height other than required by Article G2.7.1.2.3.

G2.7.1.2.3 The minimum geometric requirements for combination railings, beyond those required to meet crash test requirements and the requirements of Article G2.7.1.2.1, shall be those required for bicycle railings or pedestrian railings, as appropriate. (See Articles G2.7.2 and G2.7.3.)

G2.7.1.3 Performance Levels and Selection Procedures

G2.7.1.3.1 Railing performance levels are described by crash test requirements. Table G2.7.1.3A

TABLE G2.7.1.3A Bridge Railing Performance Levels and Crash Test Criteria

PERFORMANCE LEVELS		TEST SPEEDS—mph[1,2] TEST VEHICLE DESCRIPTIONS AND IMPACT ANGLES: Small Automobile W = 1.8 Kips A = 5.4′ ± 0.1′ B = 5.5′ H_{cg} = 20″ ± 1″ θ = 20 deg.	Pickup Truck W = 5.4 Kips A = 8.5′ ± 0.1′ B = 6.5′ H_{cg} = 27″ ± 1″ θ = 20 deg.	Medium Single-Unit Truck W = 18.0 Kips A = 12.8′ ± 0.2′ B = 7.5′ H_{cg} = 49″ ± 1″ θ = 15 deg.	Van-Type Tractor-Trailer[4] W = 50.0 Kips A = 12.5′ ± 0.5′ B = 8.0′ H_{cg} = See Note 4 R = 0.61 ± 0.01 θ = 15 deg.
PL-1		50	45		
PL-2		60	60	50	
PL-3		60	60		50
CRASH TEST EVALUATION CRITERIA[3]	Required	a, b, c, d, g	a, b, c, d	a, b, c	a, b, c
	Desirable[5]	e, f, h	e, f, g, h	d, e, f, h	d, e, f, h

Notes:

1. Except as noted, all full-scale tests shall be conducted and reported in accordance with the requirements in NCHRP Report No. 230. In addition, the maximum loads that can be transmitted from the bridge railing to the bridge deck are to be determined from static force measurements or ultimate strength analysis and reported.
2. Permissible tolerances on the test speeds and angles are as follows:

Speed	−1.0 mph	+2.5 mph
Angle	−1.0 deg.	+2.5 deg.

Tests that indicate acceptable railing performance but that exceed the allowable upper tolerances will be accepted.

3. Criteria for evaluating bridge railing crash test results are as follows:
   a. The test article shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the test article is acceptable.
   b. Detached elements, fragments, or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
   c. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.
   d. The vehicle shall remain upright during and after collision.
   e. The test article shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle or, in the case of a combination vehicle, the rear of the tractor or trailer does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.
   f. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction, μ:

μ	Assessment
0–0.25	Good
0.26–0.35	Fair
>0.35	Marginal

where $\mu = (\cos\theta - V_p/V)/\sin\theta$

TABLE G2.7.1.3A (Continued) Bridge Railing Performance Levels and Crash Test Criteria

g. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 2.0-ft. longitudinal and 1.0-ft. lateral diplacements, shall be less than:

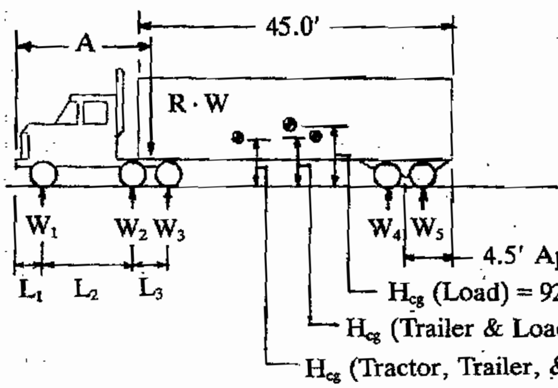
Occupant Impact Velocity–fps	
Longitudinal	Lateral
30	25

and the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than:

Occupant Ridedown Acceleration—g's	
Longitudinal	Lateral
15	15

h. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 100 ft. plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 20-ft. from the line of the traffic face of the railing. The brakes shall not be applied until the vehicle has traveled at least 100-ft. plus the length of the test vehicle from the point of initial impact.

4. Values A and R are estimated values describing the test vehicle and its loading. Values of A and R are described in the figure below and calculated as follows:



Min. Load = 20.5 Kips

$L_1 = 30'' \pm 1''$

$L_2 + \frac{L_3}{2} = 169'' \pm 4''$

$$A = L_1 + \frac{W_2L_2 + W_3(L_2 + L_3)}{W_1 + W_2 + W_3}$$

$$R = \frac{W_1 + W_2 + W_3}{W}$$

$W = W_1 + W_2 + W_3 + W_4 + W_5$
= total vehicle weight.

5. Test articles that do not meet the desirable evaluation criteria shall have their performance evaluated by a designated authority that will decide whether the test article is likely to meet its intended use requirements.

lists bridge railing performance levels and associated crash tests to be used in developing and qualifying railings.

G2.7.1.3.2 Unless a more exact method is used, Table G2.7.1.3B shall be used to estimate the appropriate performance level for a bridge railing. Values given in Table G2.7.1.3B are for bridges on tangent, level roadways, with deck surfaces approximately 35 feet above the under structure ground or water surface, and with low occupancy land use or shallow water under the structure. The traffic volume to be used to determine the appropriate performance level for a bridge railing is to be based on the estimated construction-year average daily traffic, provided this traffic includes that which will be contributed by any soon to be completed parts of the highway network or land development. For bridges carrying other than tangent, level roadways or with heights or understructure conditions that differ from those upon which Table G2.7.1.3B is based, the traffic volume used to determine an appropriate bridge railing performance level shall be the estimated construction-year traffic volume adjusted by correction factors given in Figures G2.7.1.3A and G2.7.1.3B.

Railing performance selection guidance in Table G2.7.1.3A assumes relatively free flowing traffic. To account for the effect traffic congestion has on traffic speeds, and thus the frequency of design level impacts on a railing, for sites with a design speed of 50 mph or greater and a construction-year ADT

TABLE G2.7.1.3B Bridge Railing Performance Level Selection Table

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd) — Highway Type		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	Divided (or Undivided with 5 or more Lanes) PERFORMANCE LEVEL PL-1 PL-2 PL-3	Undivided with 4 Lanes or Less PERFORMANCE LEVEL PL-1 PL-2 PL-3	One Way PERFORMANCE LEVEL PL-1 PL-2 PL-3
30	0	0– 3	0 to 151.0 to ∞	0 to 144.3 to ∞	0 to 75.5 to ∞
30	0	3– 7	0 to 283.2 to ∞	0 to 265.2 to ∞	0 to 141.6 to ∞
30	0	7–12	0 to ∞	0 to ∞	0 to 316.1 to ∞
30	0	>12	0 to ∞	0 to ∞	0 to ∞
30	5	0– 3	0 to 56.6 to ∞	0 to 48.0 to ∞	0 to 28.3 to 357.1 to ∞
30	5	3– 7	0 to 90.4 to ∞	0 to 74.6 to ∞	0 to 45.2 to ∞
30	5	7–12	0 to 148.3 to ∞	0 to 128.9 to ∞	0 to 74.2 to ∞
30	5	>12	0 to 316.0 to ∞	0 to 277.9 to ∞	0 to 158.0 to ∞
30	10	0– 3	0 to 23.9 to 179.8 to ∞	0 to 19.3 to 147.9 to ∞	0 to 12.0 to 89.9 to ∞
30	10	3– 7	0 to 36.5 to 258.3 to ∞	0 to 28.8 to 228.7 to ∞	0 to 18.3 to 129.2 to ∞
30	10	7–12	0 to 55.9 to 404.4 to ∞	0 to 46.5 to 364.6 to ∞	0 to 28.0 to 202.2 to ∞
30	10	>12	0 to 100.7 to ∞	0 to 84.6 to ∞	0 to 50.4 to 417.1 to ∞
30	15	0– 3	0 to 15.1 to 102.9 to ∞	0 to 12.1 to 84.5 to ∞	0 to 7.6 to 51.5 to ∞
30	15	3– 7	0 to 22.8 to 146.6 to ∞	0 to 17.9 to 129.2 to ∞	0 to 11.4 to 73.3 to ∞
30	15	7–12	0 to 34.4 to 228.5 to ∞	0 to 28.3 to 205.3 to ∞	0 to 17.2 to 114.3 to ∞
30	15	>12	0 to 59.9 to 472.0 to ∞	0 to 49.9 to 466.5 to ∞	0 to 30.0 to 236.0 to ∞
30	20	0– 3	0 to 11.1 to 72.0 to ∞	0 to 8.8 to 59.1 to ∞	0 to 5.6 to 36.0 to ∞
30	20	3– 7	0 to 16.6 to 102.4 to ∞	0 to 13.0 to 90.0 to ∞	0 to 8.3 to 51.2 to ∞
30	20	7–12	0 to 24.9 to 159.2 to ∞	0 to 20.4 to 142.9 to ∞	0 to 12.5 to 79.6 to ∞
30	20	>12	0 to 42.6 to 329.1 to ∞	0 to 35.4 to 325.2 to ∞	0 to 21.3 to 164.6 to ∞
30	25	0– 3	0 to 8.7 to 55.4 to ∞	0 to 6.9 to 45.4 to ∞	0 to 4.4 to 27.7 to ∞
30	25	3– 7	0 to 13.1 to 78.6 to ∞	0 to 10.2 to 69.1 to ∞	0 to 6.6 to 39.3 to ∞
30	25	7–12	0 to 19.5 to 122.2 to ∞	0 to 15.9 to 109.6 to ∞	0 to 9.8 to 61.1 to ∞
30	25	>12	0 to 33.1 to 252.6 to ∞	0 to 27.4 to 249.6 to ∞	0 to 16.6 to 126.3 to ∞
30	30	0– 3	0 to 7.2 to 45.0 to ∞	0 to 5.7 to 36.9 to ∞	0 to 3.6 to 22.5 to ∞
30	30	3– 7	0 to 10.8 to 63.8 to ∞	0 to 8.4 to 56.1 to ∞	0 to 5.4 to 31.9 to ∞
30	30	7–12	0 to 16.0 to 99.1 to ∞	0 to 13.1 to 88.8 to ∞	0 to 8.0 to 49.6 to ∞
30	30	>12	0 to 27.0 to 205.0 to ∞	0 to 22.4 to 202.5 to ∞	0 to 13.5 to 102.5 to ∞
30	35	0– 3	0 to 6.1 to 37.9 to ∞	0 to 4.8 to 31.1 to ∞	0 to 3.1 to 19.0 to ∞
30	35	3– 7	0 to 9.2 to 53.7 to ∞	0 to 7.1 to 47.2 to ∞	0 to 4.6 to 26.9 to ∞
30	35	7–12	0 to 13.6 to 83.4 to ∞	0 to 11.1 to 74.7 to ∞	0 to 6.8 to 41.7 to ∞
30	35	>12	0 to 22.8 to 172.5 to ∞	0 to 18.9 to 170.4 to ∞	0 to 11.4 to 86.3 to ∞
30	40	0– 3	0 to 5.3 to 32.8 to ∞	0 to 4.2 to 26.8 to ∞	0 to 2.7 to 16.4 to ∞
30	40	3– 7	0 to 8.0 to 46.4 to ∞	0 to 6.2 to 40.7 to ∞	0 to 4.0 to 23.2 to ∞
30	40	7–12	0 to 11.8 to 72.0 to ∞	0 to 9.6 to 64.5 to ∞	0 to 5.9 to 36.0 to ∞
30	40	>12	0 to 19.8 to 148.9 to ∞	0 to 16.3 to 147.1 to ∞	0 to 9.9 to 74.5 to ∞

See Notes at the end of the Table.

TABLE G2.7.1.3B (Continued) Bridge Railing Performance Level Selection Table

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)								
			Highway Type								
			Divided (or Undivided with 5 or more Lanes)			Undivided with 4 Lanes or Less			One Way		
			PERFORMANCE LEVEL			PERFORMANCE LEVEL			PERFORMANCE LEVEL		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3
40	0	0–3	0 to	19.0 to	∞	0 to	14.4 to	∞	0 to	9.5 to	∞
40	0	3–7	0 to	24.8 to	∞	0 to	19.0 to	∞	0 to	12.4 to	∞
40	0	7–12	0 to	33.1 to	∞	0 to	27.2 to	∞	0 to	16.6 to	∞
40	0	<12	0 to	59.3 to	∞	0 to	51.1 to	∞	0 to	29.7 to	∞
40	5	0–3	0 to	14.0 to	280.7 to ∞	0 to	10.4 to	202.4 to ∞	0 to	7.0 to	140.4 to ∞
40	5	3–7	0 to	18.0 to	335.1 to ∞	0 to	13.4 to	253.8 to ∞	0 to	9.0 to	167.6 to ∞
40	5	7–12	0 to	24.4 to	452.0 to ∞	0 to	19.2 to	366.7 to ∞	0 to	12.2 to	226.0 to ∞
40	5	>12	0 to	39.5 to	∞	0 to	32.1 to	∞	0 to	19.8 to	362.7 to ∞
40	10	0–3	0 to	9.8 to	79.7 to ∞	0 to	7.1 to	55.6 to ∞	0 to	4.9 to	39.9 to ∞
40	10	3–7	0 to	12.7 to	89.8 to ∞	0 to	9.2 to	68.6 to ∞	0 to	6.4 to	44.9 to ∞
40	10	7–12	0 to	16.9 to	132.4 to ∞	0 to	12.8 to	102.3 to ∞	0 to	8.5 to	66.2 to ∞
40	10	>12	0 to	25.8 to	183.6 to ∞	0 to	20.1 to	157.2 to ∞	0 to	12.9 to	91.8 to ∞
40	15	0–3	0 to	7.5 to	46.4 to ∞	0 to	5.4 to	32.2 to ∞	0 to	3.8 to	23.2 to ∞
40	15	3–7	0 to	9.8 to	51.9 to ∞	0 to	7.0 to	39.6 to ∞	0 to	4.9 to	26.0 to ∞
40	15	7–12	0 to	12.9 to	77.6 to ∞	0 to	9.6 to	59.4 to ∞	0 to	6.5 to	38.8 to ∞
40	15	>12	0 to	19.1 to	105.1 to ∞	0 to	14.6 to	89.6 to ∞	0 to	9.6 to	52.6 to ∞
40	20	0–3	0 to	6.1 to	32.8 to ∞	0 to	4.4 to	22.7 to ∞	0 to	3.1 to	16.4 to ∞
40	20	3–7	0 to	8.0 to	36.5 to ∞	0 to	5.6 to	27.9 to ∞	0 to	4.0 to	18.3 to ∞
40	20	7–12	0 to	10.4 to	54.9 to ∞	0 to	7.7 to	41.9 to ∞	0 to	5.2 to	27.5 to ∞
40	20	>12	0 to	15.2 to	73.6 to ∞	0 to	11.5 to	62.7 to ∞	0 to	7.6 to	36.8 to ∞
40	25	0–3	0 to	5.1 to	25.3 to ∞	0 to	3.6 to	17.5 to ∞	0 to	2.6 to	12.7 to ∞
40	25	3–7	0 to	6.7 to	28.1 to ∞	0 to	4.7 to	21.5 to ∞	0 to	3.4 to	14.1 to ∞
40	25	7–12	0 to	8.8 to	42.4 to ∞	0 to	6.4 to	32.3 to ∞	0 to	4.4 to	21.2 to ∞
40	25	>12	0 to	12.6 to	56.7 to ∞	0 to	9.5 to	48.2 to ∞	0 to	6.3 to	28.4 to ∞
40	30	0–3	0 to	4.4 to	20.6 to ∞	0 to	3.1 to	14.2 to ∞	0 to	2.2 to	10.3 to ∞
40	30	3–7	0 to	5.8 to	22.9 to ∞	0 to	4.1 to	17.5 to ∞	0 to	2.9 to	11.5 to ∞
40	30	7–12	0 to	7.5 to	34.6 to ∞	0 to	5.5 to	26.3 to ∞	0 to	3.8 to	17.3 to ∞
40	30	>12	0 to	10.8 to	46.1 to ∞	0 to	8.0 to	39.1 to ∞	0 to	5.4 to	23.1 to ∞
40	35	0–3	0 to	3.9 to	17.4 to ∞	0 to	2.8 to	12.0 to ∞	0 to	2.0 to	8.7 to ∞
40	35	3–7	0 to	5.1 to	19.3 to ∞	0 to	3.6 to	14.7 to ∞	0 to	2.6 to	9.7 to ∞
40	35	7–12	0 to	6.6 to	29.2 to ∞	0 to	4.8 to	22.2 to ∞	0 to	3.3 to	14.6 to ∞
40	35	>12	0 to	9.4 to	38.8 to ∞	0 to	7.0 to	32.9 to ∞	0 to	4.7 to	19.4 to ∞
40	40	0–3	0 to	3.5 to	15.0 to ∞	0 to	2.5 to	10.4 to ∞	0 to	1.8 to	7.5 to ∞
40	40	3–7	0 to	4.6 to	16.7 to ∞	0 to	3.2 to	12.7 to ∞	0 to	2.3 to	8.4 to ∞
40	40	7–12	0 to	5.9 to	25.3 to ∞	0 to	4.2 to	19.2 to ∞	0 to	3.0 to	12.7 to ∞
40	40	>12	0 to	8.4 to	33.5 to ∞	0 to	6.2 to	28.4 to ∞	0 to	4.2 to	16.8 to ∞

See Notes at the end of the Table.

TABLE G2.7.1.3B (Continued) Bridge Railing Performance Level Selection Table

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)								
			Highway Type								
			Divided (or Undivided with 5 or more Lanes)			Undivided with 4 Lanes or Less			One Way		
			PERFORMANCE LEVEL			PERFORMANCE LEVEL			PERFORMANCE LEVEL		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3
50	0	0–3	0 to	6.2 to	∞	0 to	4.2 to	∞	0 to	3.1 to	∞
50	0	3–7	0 to	7.2 to	∞	0 to	5.0 to	∞	0 to	3.6 to	∞
50	0	7–12	0 to	9.9 to	∞	0 to	7.3 to	∞	0 to	5.0 to	∞
50	0	<12	0 to	13.0 to	∞	0 to	9.6 to	∞	0 to	6.5 to	∞
50	5	0–3	0 to	5.5 to	162.2 to ∞	0 to	3.7 to	107.0 to ∞	0 to	2.8 to	81.1 to ∞
50	5	3–7	0 to	6.3 to	188.6 to ∞	0 to	4.4 to	134.1 to ∞	0 to	3.2 to	94.3 to ∞
50	5	7–12	0 to	8.4 to	247.3 to ∞	0 to	6.1 to	171.9 to ∞	0 to	4.2 to	123.7 to ∞
50	5	>12	0 to	11.2 to	314.7 to ∞	0 to	8.2 to	245.4 to ∞	0 to	5.6 to	157.4 to ∞
50	10	0–3	0 to	4.7 to	50.0 to ∞	0 to	3.2 to	32.0 to ∞	0 to	2.4 to	25.0 to ∞
50	10	3–7	0 to	5.4 to	61.4 to ∞	0 to	3.7 to	41.8 to ∞	0 to	2.7 to	30.7 to ∞
50	10	7–12	0 to	7.2 to	70.6 to ∞	0 to	5.1 to	49.3 to ∞	0 to	3.6 to	35.3 to ∞
50	10	>12	0 to	9.6 to	88.5 to ∞	0 to	6.9 to	67.8 to ∞	0 to	4.8 to	44.3 to ∞
50	15	0–3	0 to	4.1 to	29.6 to ∞	0 to	2.8 to	18.8 to ∞	0 to	2.1 to	14.8 to ∞
50	15	3–7	0 to	4.8 to	36.7 to ∞	0 to	3.3 to	24.8 to ∞	0 to	2.4 to	18.4 to ∞
50	15	7–12	0 to	6.3 to	41.2 to ∞	0 to	4.4 to	28.8 to ∞	0 to	3.2 to	20.6 to ∞
50	15	>12	0 to	8.4 to	51.5 to ∞	0 to	5.9 to	39.4 to ∞	0 to	4.2 to	25.8 to ∞
50	20	0–3	0 to	3.7 to	21.0 to ∞	0 to	2.5 to	13.3 to ∞	0 to	1.9 to	10.5 to ∞
50	20	3–7	0 to	4.3 to	26.1 to ∞	0 to	2.9 to	17.6 to ∞	0 to	2.2 to	13.1 to ∞
50	20	7–12	0 to	5.6 to	29.1 to ∞	0 to	3.9 to	20.3 to ∞	0 to	2.8 to	14.6 to ∞
50	20	>12	0 to	7.5 to	36.3 to ∞	0 to	5.2 to	27.7 to ∞	0 to	3.8 to	18.2 to ∞
50	25	0–3	0 to	3.3 to	16.3 to ∞	0 to	2.2 to	10.3 to ∞	0 to	1.7 to	8.2 to ∞
50	25	3–7	0 to	3.9 to	20.3 to ∞	0 to	2.6 to	13.7 to ∞	0 to	2.0 to	10.2 to ∞
50	25	7–12	0 to	5.0 to	22.5 to ∞	0 to	3.5 to	15.7 to ∞	0 to	2.5 to	11.3 to ∞
50	25	>12	0 to	6.7 to	28.1 to ∞	0 to	4.7 to	21.4 to ∞	0 to	3.4 to	14.1 to ∞
50	30	0–3	0 to	3.1 to	13.3 to ∞	0 to	2.0 to	8.4 to ∞	0 to	1.6 to	6.7 to ∞
50	30	3–7	0 to	3.5 to	16.6 to ∞	0 to	2.4 to	1.1 to ∞	0 to	1.8 to	8.3 to ∞
50	30	7–12	0 to	4.5 to	18.3 to ∞	0 to	3.1 to	12.8 to ∞	0 to	2.3 to	9.2 to ∞
50	30	>12	0 to	6.1 to	22.9 to ∞	0 to	4.2 to	17.4 to ∞	0 to	3.1 to	11.5 to ∞
50	35	0–3	0 to	2.8 to	11.2 to ∞	0 to	1.9 to	7.1 to ∞	0 to	1.4 to	5.6 to ∞
50	35	3–7	0 to	3.2 to	14.0 to ∞	0 to	2.2 to	9.4 to ∞	0 to	1.6 to	7.0 to ∞
50	35	7–12	0 to	4.2 to	15.5 to ∞	0 to	2.9 to	10.8 to ∞	0 to	2.1 to	7.8 to ∞
50	35	>12	0 to	5.6 to	19.3 to ∞	0 to	3.8 to	14.7 to ∞	0 to	2.8 to	9.7 to ∞
50	40	0–3	0 to	2.6 to	9.7 to ∞	0 to	1.7 to	6.1 to ∞	0 to	1.3 to	4.9 to ∞
50	40	3–7	0 to	3.0 to	12.2 to ∞	0 to	2.0 to	8.2 to ∞	0 to	1.5 to	6.1 to ∞
50	40	7–12	0 to	3.8 to	13.4 to ∞	0 to	2.6 to	9.3 to ∞	0 to	1.9 to	6.7 to ∞
50	40	>12	0 to	5.2 to	16.7 to ∞	0 to	3.5 to	12.7 to ∞	0 to	2.6 to	8.4 to ∞

See Notes at the end of the Table.

TABLE G2.7.1.3B (Continued) Bridge Railing Performance Level Selection Table

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)								
			Highway Type								
			Divided (or Undivided with 5 or more Lanes)			Undivided with 4 Lanes or Less			One Way		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	PERFORMANCE LEVEL PL-1	PL-2	PL-3	PERFORMANCE LEVEL PL-1	PL-2	PL-3	PERFORMANCE LEVEL PL-1	PL-2	PL-3
60	0	0–3	0 to	3.2 to	∞	0 to	2.0 to	∞	0 to	1.6 to	∞
60	0	3–7	0 to	3.6 to	∞	0 to	2.3 to	∞	0 to	1.8 to	∞
60	0	7–12	0 to	4.4 to	∞	0 to	2.9 to	∞	0 to	2.2 to	∞
60	0	<12	0 to	5.5 to	∞	0 to	3.5 to	∞	0 to	2.8 to	∞
60	5	0–3	0 to	3.0 to	107.3 to ∞	0 to	1.9 to	70.3 to ∞	0 to	1.5 to	53.7 to ∞
60	5	3–7	0 to	3.3 to	126.3 to ∞	0 to	2.1 to	82.8 to ∞	0 to	1.7 to	63.2 to ∞
60	5	7–12	0 to	4.1 to	158.4 to ∞	0 to	2.7 to	105.6 to ∞	0 to	2.1 to	79.2 to ∞
60	5	>12	0 to	5.0 to	203.8 to ∞	0 to	3.3 to	138.2 to ∞	0 to	2.5 to	101.9 to ∞
60	10	0–3	0 to	2.8 to	39.6 to ∞	0 to	1.8 to	25.0 to ∞	0 to	1.4 to	19.8 to ∞
60	10	3–7	0 to	3.1 to	47.5 to ∞	0 to	2.0 to	29.3 to ∞	0 to	1.6 to	23.8 to ∞
60	10	7–12	0 to	3.9 to	53.1 to ∞	0 to	2.5 to	33.7 to ∞	0 to	2.0 to	26.6 to ∞
60	10	>12	0 to	4.7 to	67.6 to ∞	0 to	3.1 to	44.1 to ∞	0 to	2.4 to	33.8 to ∞
60	15	0–3	0 to	2.7 to	24.3 to ∞	0 to	1.7 to	15.2 to ∞	0 to	1.4 to	12.2 to ∞
60	15	3–7	0 to	2.9 to	29.3 to ∞	0 to	1.9 to	17.8 to ∞	0 to	1.5 to	14.7 to ∞
60	15	7–12	0 to	3.7 to	31.9 to ∞	0 to	2.4 to	20.0 to ∞	0 to	1.9 to	16.0 to ∞
60	15	>12	0 to	4.5 to	40.5 to ∞	0 to	2.9 to	26.2 to ∞	0 to	2.3 to	20.3 to ∞
60	20	0–3	0 to	2.5 to	17.5 to ∞	0 to	1.6 to	10.9 to ∞	0 to	1.3 to	8.8 to ∞
60	20	3–7	0 to	2.8 to	21.1 to ∞	0 to	1.8 to	12.8 to ∞	0 to	1.4 to	10.6 to ∞
60	20	7–12	0 to	3.5 to	22.8 to ∞	0 to	2.2 to	14.3 to ∞	0 to	1.8 to	11.4 to ∞
60	20	>12	0 to	4.2 to	28.9 to ∞	0 to	2.8 to	18.7 to ∞	0 to	2.1 to	14.5 to ∞
60	25	0–3	0 to	2.4 to	13.7 to ∞	0 to	1.5 to	8.5 to ∞	0 to	1.2 to	6.9 to ∞
60	25	3–7	0 to	2.6 to	16.5 to ∞	0 to	1.7 to	10.0 to ∞	0 to	1.3 to	8.3 to ∞
60	25	7–12	0 to	3.3 to	17.7 to ∞	0 to	2.1 to	11.1 to ∞	0 to	1.7 to	8.9 to ∞
60	25	>12	0 to	4.0 to	22.5 to ∞	0 to	2.6 to	14.5 to ∞	0 to	2.0 to	11.3 to ∞
60	30	0–3	0 to	2.3 to	11.2 to ∞	0 to	1.4 to	7.0 to ∞	0 to	1.2 to	5.6 to ∞
60	30	3–7	0 to	2.5 to	13.6 to ∞	0 to	1.6 to	8.2 to ∞	0 to	1.3 to	6.8 to ∞
60	30	7–12	0 to	3.2 to	14.5 to ∞	0 to	2.0 to	9.0 to ∞	0 to	1.6 to	7.3 to ∞
60	30	>12	0 to	3.8 to	18.4 to ∞	0 to	2.5 to	11.9 to ∞	0 to	1.9 to	9.2 to ∞
60	35	0–3	0 to	2.2 to	9.5 to ∞	0 to	1.4 to	5.9 to ∞	0 to	1.1 to	4.8 to ∞
60	35	3–7	0 to	2.4 to	11.5 to ∞	0 to	1.5 to	6.9 to ∞	0 to	1.2 to	5.8 to ∞
60	35	7–12	0 to	3.0 to	12.3 to ∞	0 to	1.9 to	7.7 to ∞	0 to	1.5 to	6.2 to ∞
60	35	>12	0 to	3.6 to	15.6 to ∞	0 to	2.4 to	10.0 to ∞	0 to	1.8 to	7.8 to ∞
60	40	0–3	0 to	2.1 to	8.3 to ∞	0 to	1.3 to	5.1 to ∞	0 to	1.1 to	4.2 to ∞
60	40	3–7	0 to	2.3 to	10.0 to ∞	0 to	1.4 to	6.0 to ∞	0 to	1.2 to	5.0 to ∞
60	40	7–12	0 to	2.9 to	10.6 to ∞	0 to	1.9 to	6.6 to ∞	0 to	1.5 to	5.3 to ∞
60	40	>12	0 to	3.5 to	13.5 to ∞	0 to	2.3 to	8.7 to ∞	0 to	1.8 to	6.8 to ∞

See Notes at the end of the Table.

TABLE G2.7.1.3B (Continued) Bridge Railing Performance Level Selection Table

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)								
			Highway Type								
			Divided (or Undivided with 5 or more Lanes)			Undivided with 4 Lanes or Less			One Way		
			PERFORMANCE LEVEL			PERFORMANCE LEVEL			PERFORMANCE LEVEL		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3	PL-1	PL-2	PL-3
70	0	0–3	0 to	2.2 to	191.4 to ∞	0 to	1.3 to	165.0 to ∞	0 to	1.1 to	95.7 to ∞
70	0	3–7	0 to	2.4 to	379.1 to ∞	0 to	1.5 to	301.5 to ∞	0 to	1.2 to	189.6 to ∞
70	0	7–12	0 to	2.8 to ∞		0 to	1.7 to	402.4 to ∞	0 to	1.4 to	256.4 to ∞
70	0	>12	0 to	3.2 to ∞		0 to	2.0 to ∞		0 to	1.6 to ∞	
70	5	0–3	0 to	2.1 to	63.1 to ∞	0 to	1.3 to	42.2 to ∞	0 to	1.1 to	31.6 to ∞
70	5	3–7	0 to	2.3 to	80.0 to ∞	0 to	1.4 to	51.6 to ∞	0 to	1.2 to	40.0 to ∞
70	5	7–12	0 to	2.7 to	96.4 to ∞	0 to	1.6 to	64.0 to ∞	0 to	1.4 to	48.2 to ∞
70	5	>12	0 to	3.1 to	127.6 to ∞	0 to	1.9 to	84.0 to ∞	0 to	1.6 to	63.8 to ∞
70	10	0–3	0 to	2.0 to	32.1 to ∞	0 to	1.2 to	20.0 to ∞	0 to	1.0 to	16.1 to ∞
70	10	3–7	0 to	2.3 to	38.5 to ∞	0 to	1.4 to	22.9 to ∞	0 to	1.2 to	19.3 to ∞
70	10	7–12	0 to	2.6 to	42.2 to ∞	0 to	1.6 to	26.7 to ∞	0 to	1.3 to	21.1 to ∞
70	10	>12	0 to	3.0 to	53.0 to ∞	0 to	1.8 to	33.1 to ∞	0 to	1.5 to	26.5 to ∞
70	15	0–3	0 to	2.0 to	21.5 to ∞	0 to	1.2 to	13.1 to ∞	0 to	1.0 to	10.8 to ∞
70	15	3–7	0 to	2.2 to	25.3 to ∞	0 to	1.3 to	14.7 to ∞	0 to	1.1 to	12.7 to ∞
70	15	7–12	0 to	2.6 to	27.0 to ∞	0 to	1.6 to	16.9 to ∞	0 to	1.3 to	13.5 to ∞
70	15	>12	0 to	3.0 to	33.5 to ∞	0 to	1.8 to	20.6 to ∞	0 to	1.5 to	16.8 to ∞
70	20	0–3	0 to	1.9 to	16.2 to ∞	0 to	1.2 to	9.7 to ∞	0 to	1.0 to	8.1 to ∞
70	20	3–7	0 to	2.1 to	18.9 to ∞	0 to	1.3 to	10.8 to ∞	0 to	1.1 to	9.5 to ∞
70	20	7–12	0 to	2.5 to	19.9 to ∞	0 to	1.5 to	12.3 to ∞	0 to	1.3 to	10.0 to ∞
70	20	>12	0 to	2.9 to	24.4 to ∞	0 to	1.8 to	15.0 to ∞	0 to	1.5 to	12.2 to ∞
70	25	0–3	0 to	1.9 to	13.0 to ∞	0 to	1.1 to	7.8 to ∞	0 to	1.0 to	6.5 to ∞
70	25	3–7	0 to	2.0 to	15.1 to ∞	0 to	1.3 to	8.6 to ∞	0 to	1.0 to	7.6 to ∞
70	25	7–12	0 to	2.5 to	15.7 to ∞	0 to	1.5 to	9.7 to ∞	0 to	1.3 to	7.9 to ∞
70	25	>12	0 to	2.8 to	19.2 to ∞	0 to	1.7 to	11.8 to ∞	0 to	1.4 to	9.6 to ∞
70	30	0–3	0 to	1.8 to	10.8 to ∞	0 to	1.1 to	6.4 to ∞	0 to	0.9 to	5.4 to ∞
70	30	3–7	0 to	2.0 to	12.5 to ∞	0 to	1.2 to	7.1 to ∞	0 to	1.0 to	6.3 to ∞
70	30	7–12	0 to	2.4 to	13.0 to ∞	0 to	1.5 to	8.0 to ∞	0 to	1.2 to	6.5 to ∞
70	30	>12	0 to	2.8 to	15.9 to ∞	0 to	1.7 to	9.7 to ∞	0 to	1.4 to	8.0 to ∞
70	35	0–3	0 to	1.8 to	9.3 to ∞	0 to	1.1 to	5.5 to ∞	0 to	0.9 to	4.7 to ∞
70	35	3–7	0 to	1.9 to	10.7 to ∞	0 to	1.2 to	6.1 to ∞	0 to	1.0 to	5.4 to ∞
70	35	7–12	0 to	2.4 to	11.1 to ∞	0 to	1.5 to	6.8 to ∞	0 to	1.2 to	5.6 to ∞
70	35	>12	0 to	2.7 to	13.5 to ∞	0 to	1.7 to	8.2 to ∞	0 to	1.4 to	6.8 to ∞
70	40	0–3	0 to	1.7 to	8.1 to ∞	0 to	1.0 to	4.8 to ∞	0 to	0.9 to	4.1 to ∞
70	40	3–7	0 to	1.9 to	9.4 to ∞	0 to	1.2 to	5.3 to ∞	0 to	1.0 to	4.7 to ∞
70	40	7–12	0 to	2.3 to	9.6 to ∞	0 to	1.4 to	5.9 to ∞	0 to	1.2 to	4.8 to ∞
70	40	>12	0 to	2.7 to	11.8 to ∞	0 to	1.6 to	7.1 to ∞	0 to	1.4 to	5.9 to ∞

See Notes at the end of the Table.

Notes for use of this Table:

Adjusted ADT = $K_c \cdot K_g \cdot K_s \cdot$ (estimated construction-year ADT)

To select bridge railing performance level:

- Calculate adjusted ADT by multiplying construction-year ADT (total for highway) by adjustment factors K_c, K_g, and K_s from Figures G2.7.1.3A and G2.7.1.3B. (The estimated construction-year ADT may be limited to 10,000 vehicles per day per lane for design speeds of 50 mph or greater, where the actual estimate exceeds that amount.)
- Locate line in table that describes site conditions (design speed, percent trucks, and bridge railing offset from traveled way).
- Move across to column describing type of highway upon which bridge is located.
- Locate adjusted ADT values in table that bracket the calculated adjusted ADT for bridge site.
- At top of column within which the calculated adjusted ADT is bracketed read the bridge railing performance level.

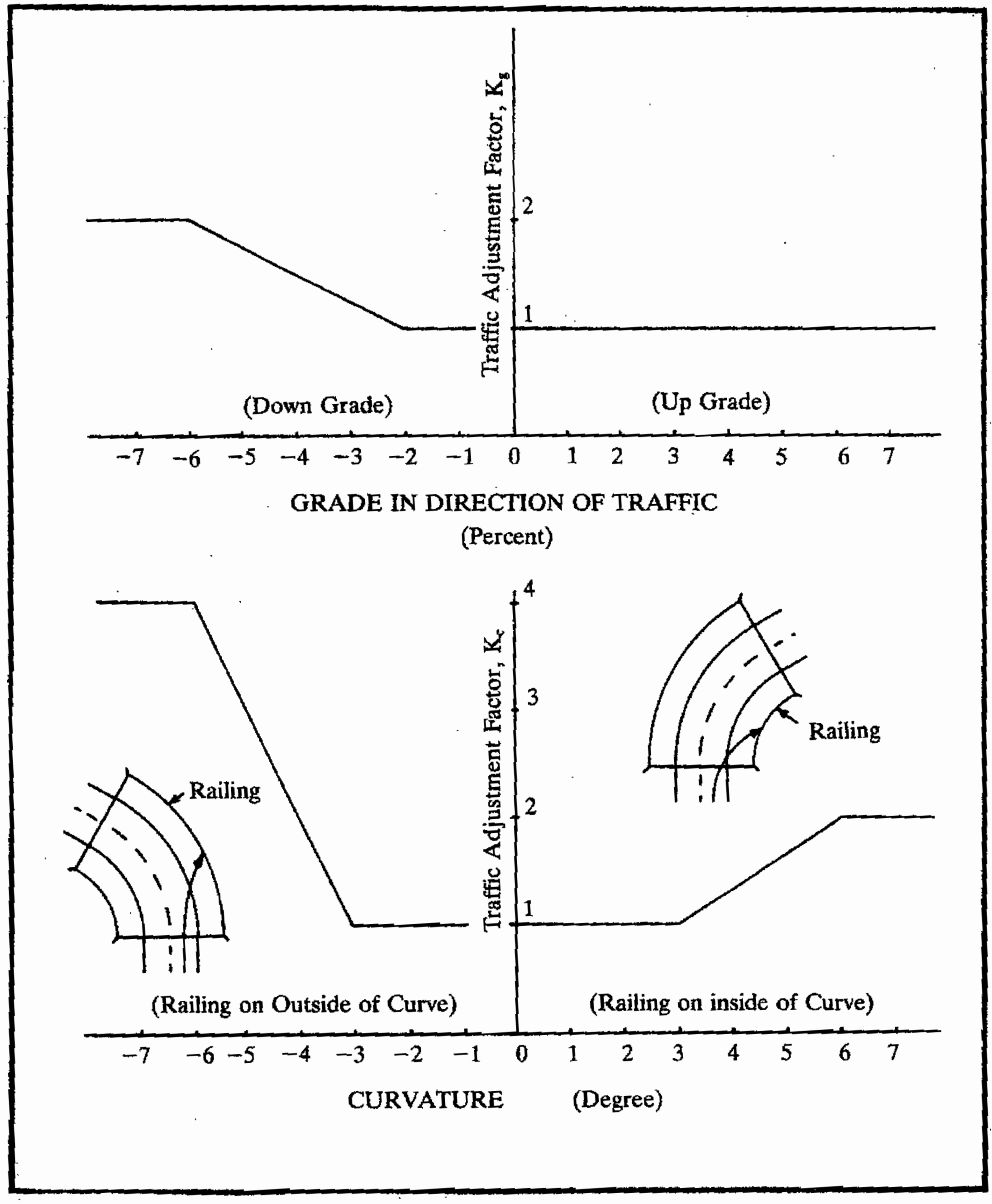


FIGURE G2.7.1.3A Grade Traffic Adjustment Factor (K_g) and Curvature Traffic Adjustment Factor (K_c) to be Applied to Estimated Construction-Year Average Daily Traffic

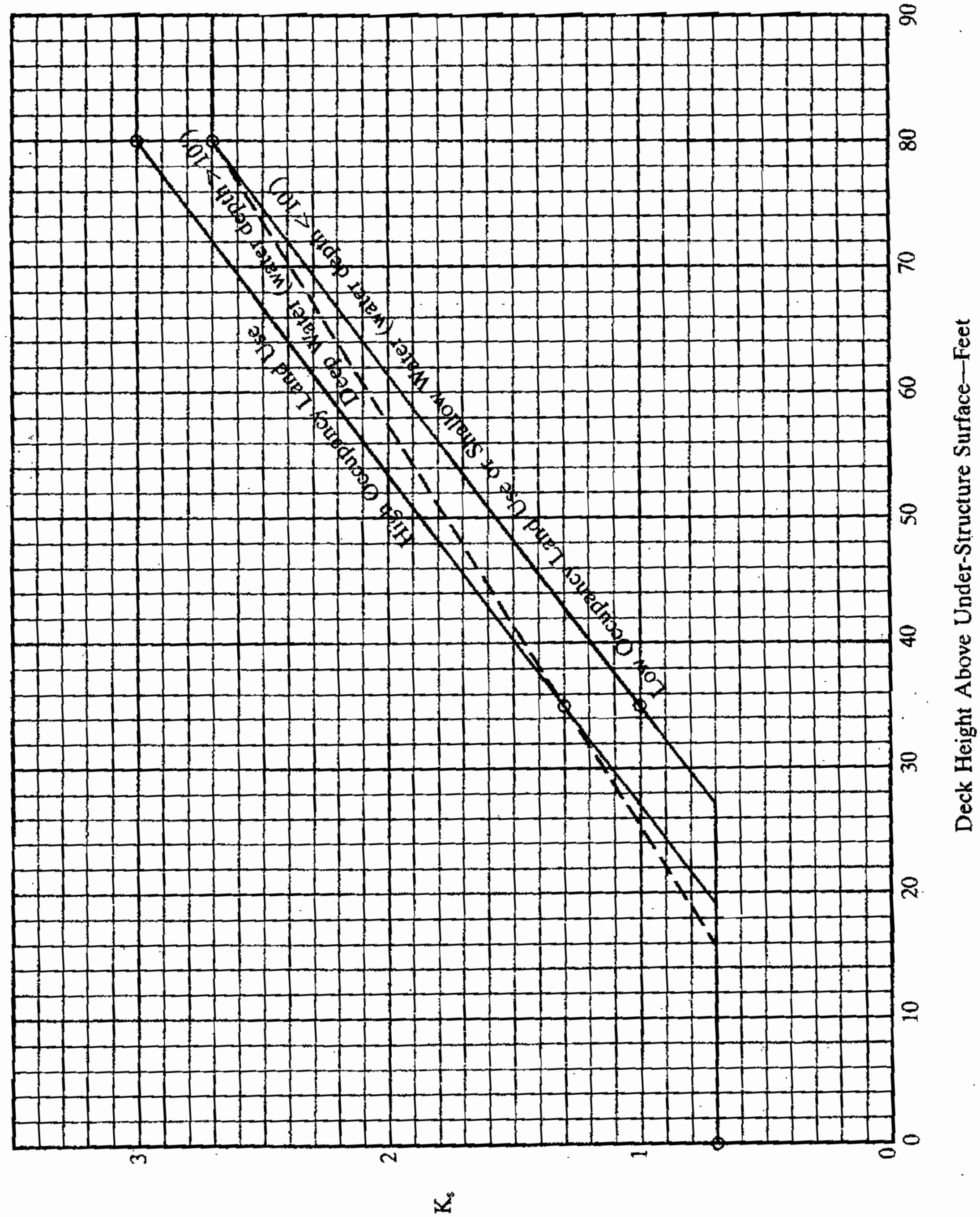


FIGURE G2.7.1.3.B Traffic Adjustment Factor (K_s) for Deck Height and Under-Structure Conditions greater than 10,000 vehicles per day per lane (vpdpl), the construction-year ADT value used in selecting a bridge railing performance level may be limited to 10,000 vpdpl.

G2.7.2 Bicycle Railing

G2.7.2.1 General

G2.7.2.1.1 Bicycle railings shall be used on bridges specifically designed to carry bicycle traffic, and on bridges where specific protection of bicyclists is deemed necessary.

G2.7.2.1.2 Railing components shall be designed with consideration to safety, appearance, and freedom of view.

G2.7.2.1.3 Materials for bicycle railing may be concrete, metal, timber, plastic, fiber reinforced plastic, or a combination thereof.

G2.7.2.2 Geometry and Loads

G2.7.2.2.1 The minimum height of a railing used to protect a bicyclist shall be 54 inches, measured from the top of the surface on which the bicycle rides to the top of the top rail.

G2.7.2.2.2 Within a band bordered by the riding surface and a line 54 inches above it, horizontal elements of the railing assembly shall have a maximum clear spacing of 15 inches. Vertical elements of the railing assembly shall have a maximum clear spacing of 8 inches. If a railing assembly employs both horizontal and vertical elements, the spacing requirements shall apply to one or the other, but not to both. Chain link fence is exempt from the rail spacing requirements listed above. In general, rails should project beyond the face of posts and/or pickets. Smooth rubrails should be attached to the railings at a height of 42 inches.

G2.7.2.2.3 The minimum design loadings for bicycle railing shall be w = 50 pounds per linear foot transversely and vertically, acting simultaneously on each rail.

G2.7.2.2.4 Design loads for rails located more than 54 inches above the riding surface shall be determined by the designer.

G2.7.2.2.5 Posts shall be designed for a transverse load of wL (where L is the post spacing) acting at the center of gravity of the upper rail, but at a height not greater than 54 inches.

G2.7.2.2.6 Refer to Figure G2.7.4 for more information concerning the application of loads.

G2.7.3 Pedestrian Railing

G2.7.3.1 General

G2.7.3.1.1 Railing components shall be designed with consideration to safety, appearance, and freedom of view.

G2.7.3.1.2 Materials for pedestrian railings may be concrete, metal, timber, plastic, fiber reinforced plastic, or a combination thereof.

G2.7.3.2 Geometry and Loads

G2.7.3.2.1 The minimum height of a pedestrian railing shall be 3 feet 6 inches measured from the top of the walkway to the top of the upper rail member.

G2.7.3.2.2 Within a band bordered by the walkway surface and a line 42 inches above it, horizontal elements of the railing assembly shall have a maximum clear spacing of 15 inches. Vertical elements of the railing assembly shall have a maximum clear spacing of 8 inches. If a railing assembly employs both horizontal and vertical elements, the spacing requirements shall apply to one or the other, but not to both. Chain link fence is exempt from the rail spacing requirements listed above. In general, rails should project beyond the face of posts and/or pickets.

G2.7.3.2.3 The minimum design loading for pedestrian railing shall be w = 50 pounds per linear foot, transversely and vertically, acting simultaneously on each longitudinal member. Rail members located more than 5 feet 0 inches above the walkway are excluded from these requirements.

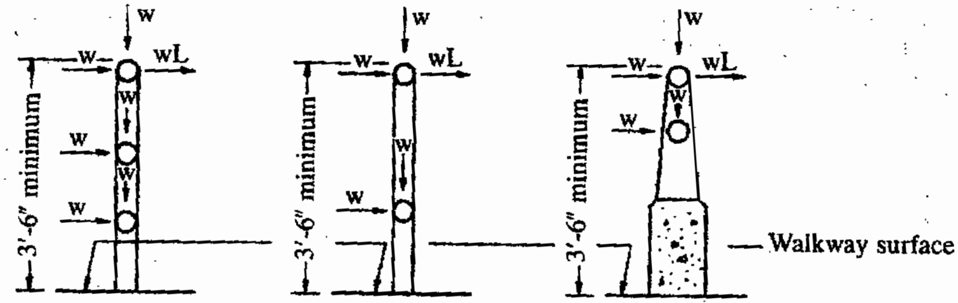
G2.7.3.2.4 Posts shall be designed for a transverse load of wL (where L is the post spacing) acting at the center of gravity of the upper rail or, for high rails, at 5 feet 0 inches maximum above the walkway.

G2.7.3.2.5 Refer to Figure G2.7.4 for more information concerning the application of loads.

G2.7.4 Structural Specifications and Guidelines for Bicycle and Pedestrian Railings

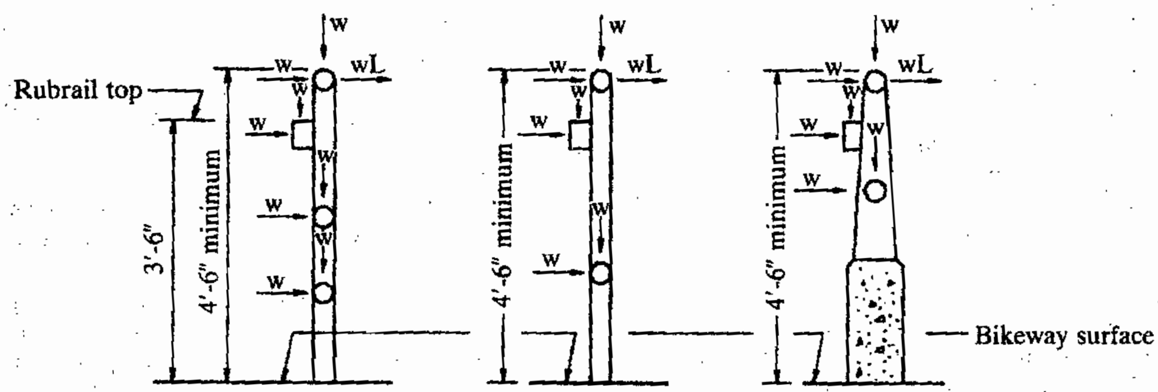
G2.7.4.1 Bicycle and Pedestrian Railings shall be designed by the elastic method to the allowable stresses for the appropriate material.

For aluminum alloys the design stresses given in the *Specifications for Aluminum Structures* Fifth Edition, December 1986, published by the Aluminum Association, Inc., for "Bridge and Similar Type Structures" for alloys 6061-T6 (Table A.6), 6351-T5 (Table A.6), and 6063-T6 (Table A.8) shall apply,



(To be used on the outer edge of a sidewalk when highway traffic is separated from pedestrian traffic by a traffic railing.)

PEDESTRIAN RAILING



(To be used on the outer edge of a bikeway when highway traffic is separated from bicycle traffic by a traffic railing.)

BICYCLE RAILING

NOTE:
If screening or solid face is presented, number of rails may be reduced; wind loads must be added if solid face is utilized.

NOTES:
1. Loadings on left are applied to rails.
2. Loads on right are applied to posts.
3. The shapes of rail members are illustrative only. Any material or combination of materials listed in Article G2.7 may be used in any configuration.

NOMENCLATURE:
w = Pedestrian or bicycle loading per unit length of rail
L = Post spacing

FIGURE G2.7.4 Pedestrian Railing, Bicycle Railing and for cast aluminum alloys the design stresses given for alloys A444.0-T4 (Table A.9), A356.0-T61 (Table A.9) and A356.0-T6 (Table A.9) shall apply.

For fabrication and welding of aluminum railing see Article 11.5 of the AASHTO *Standard Specifications for Highway Bridges.*

G2.7.4.2 The allowable unit stresses for steel shall be as given in Article 10.32 of the AASHTO *Standard Specifications for Highway Bridges*, except as modified below.

For steels not generally covered by the "Standard Specifications," but having a guaranteed yield strength, F_y, the allowable unit stress, shall be derived by applying the general formulas as given in the "Standard Specifications" under "Unit Stresses" except as indicated below.

The allowable unit stress for shear shall be $F_v = 0.33F_y$.

Round or oval steel tubes may be proportioned using an allowable bending stress, $F_b = 0.66F_y$, provided the R/t ratio (radius/thickness) is less than or equal to 40.

Square and rectangular steel tubes and steel W and I sections in bending with tension and compression on extreme fibers of laterally supported compact sections having an axis of symmetry in the plane of loading may be designed for an allowable stress $F_b = 0.60F_y$.

G2.7.4.3 The requirements for a compact section are as follows:

(a) The width to thickness ratio of projecting elements of the compression flange of W and I sections shall not exceed

$$\frac{b}{t} \leq \frac{1600}{\sqrt{F_y}} \qquad (2\text{-}1)$$

(b) The width to thickness ratio of the compression flange of square or rectangular tubes shall not exceed

$$\frac{b}{t} \leq \frac{6000}{\sqrt{F_y}} \qquad (2\text{-}2)$$

(c) The D/t ratio of webs shall not exceed

$$\frac{D}{t} \leq \frac{13000}{\sqrt{F_y}} \qquad (2\text{-}3)$$

(d) If subject to combined axial force and bending, the D/t ratio of webs shall not exceed

$$\frac{D}{t} < \frac{13{,}300\left[1 - 1.43\left(\frac{f_a}{F_a}\right)\right]}{\sqrt{F_y}} \qquad (2\text{-}4)$$

but need not be less than

$$\frac{D}{t} < \frac{7000}{\sqrt{F_y}} \qquad (2\text{-}5)$$

(e) The distance between lateral supports in inches of W or I sections shall not exceed

$$\leq \frac{2400b}{\sqrt{F_y}} \qquad (2\text{-}6)$$

or

$$\leq \frac{20{,}000{,}000\ A_f}{dF_y} \qquad (2\text{-}7)$$

G3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABS

G3.24.5 Cantilever Slabs

G3.24.5.2 Railing Loads on Bridge Decks

Railing loads applied to the bridge deck slab shall be based on the ultimate strength of the railing used (See Note 1 in Table G2.7.1.3A). Loads shall be applied and the deck designed in a manner to assure the ultimate strength of the slab will exceed that required to resist the maximum bending, shear, and punching loads that can be transmitted through the bridge railing, along with simultaneously applied wheel loads.

APPENDIX A

APPENDIX A
BRIDGE RAILING DESIGN GUIDELINES

INTRODUCTION

Ultimately, the goal of a bridge railing designer should be to reduce death and injury to the occupants of errant vehicles and to protect lives and property on, adjacent to, or below a bridge. The best way of ensuring that a railing will meet the functional needs of a site is to subject it to pertinent crash tests and evaluate the results against performance criteria that will indicate the railing's suitability. This is the objective of the test requirements, evaluation criteria, and railing performance level selection procedures given in the *Guide Specifications for Bridge Railings* to which this appendix is attached. However, crash testing is expensive and, as a result, can not be extensive enough to reveal all possible problems with a railing.

Fortunately, there has been enough railing testing and analysis to allow some confidence in designing railings to meet given performance criteria. The design guidelines that follow provide guidance for the design of railings for the three performance levels given in the guide specifications, plus two optional levels. One optional level is offered for consideration where truck volumes and highway alignment combine to produce site conditions the designer believe justify a performance level greater than a PL-3. The other optional level is offered as a virtually unbreachable railing for those locations where policy decisions are made that such a railing is to be installed.

These guidelines are intended as a basis for the design of prototype railings that are to be crash tested and for the design of one-of-a-kind railings where the cost of a crash test program may not be justified. It is also hoped they will keep designers from unwittingly reducing the effectiveness of crash-proven railing designs when making inevitable alterations to meet unique field conditions.

The guidelines are for relatively rigid railings. (Flexible railings offer some potential advantages over rigid railings, particularly by reducing loads on vehicles and railings. However, they are more complex to design and have had little acceptance to date.) At all performance levels the guidelines are intended to guard against bumper, wheel, and hood snagging for automobiles and to provide stable post-impact trajectories for automobiles, light trucks, and passenger vans. Large trucks and buses receive increasing attention in the guidelines with increasing performance level. The large van-type semi-trailer truck is a vehicle of significant concern at the PL-2 level and of major concern at the PL-3 level and optional PL-4 levels. Other vehicles are also considered important but the prevalence and accident record of the van-type semi-trailer truck justifies special attention; designing for it ensures good performance with most other large vehicles. At the optional PL-4T level additional attention is given to the containment and stable redirection of tank trucks.

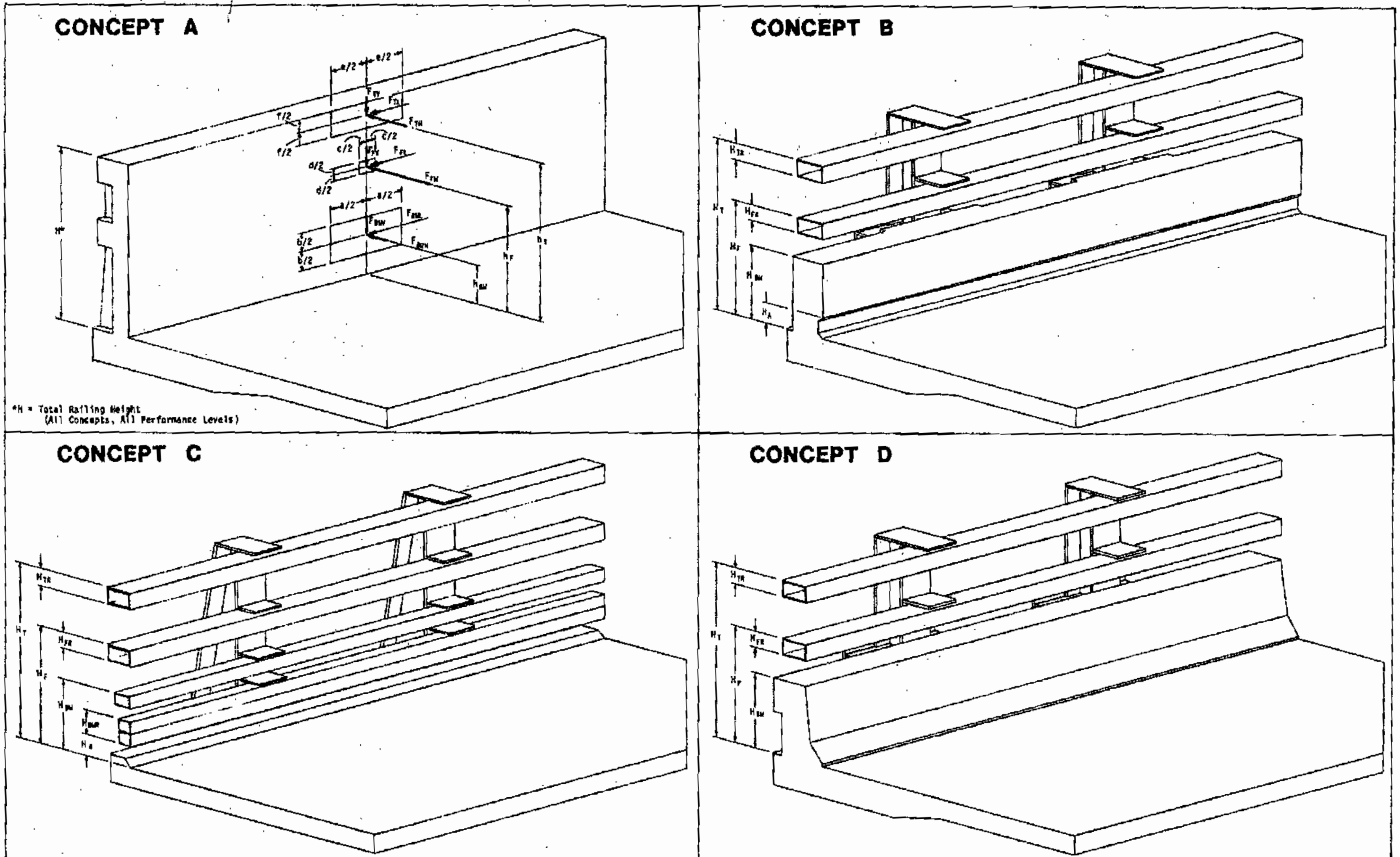
RAILING GEOMETRY

Figure A1 shows four conceptual railing layouts for the optional PL-4T performance level. Railing dimensions are given in Table A1 for the optional PL-4T and all other performance levels. Note that Figure A1 also becomes applicable to the other performance levels by assuming that railing elements that are not dimensioned in Table A1 do not exist.

The traffic face of all railings should be smooth and continuous. The clearance from the face-of-rail to the face-of-post or other potential snag point in a railing system must be great enough to preclude contact by substantial vehicle parts that might penetrate an opening in the railing. The railing configurations and dimensions suggested in Figure A1 and Table A1 are intended to block any of the typical snag offenders such as automobile bumpers, wheels, and hoods and semi-trailer floors. However, because of the great variety of vehicles on our highways, a minimum clearance of 10 inches is suggested with the railing configurations shown and is recommended as an absolute minimum for any railing configuration where snagging is an obvious possibility.

At locations where the transition from an open-face railing to a closed-face railing or a parapet creates a snag potential, the closed-face railing should be flared and overlapped by the open-face railing. The flare should be no more abrupt than 3.5 longitudinal to 1 lateral and should begin a minimum of 10 inches back of the traffic face of the open-face railing.

Post spacings in beam-and-post railings should not exceed 10 feet (see section on railing loads for discussion.)



See Table A1 for Force and Dimension Quantities.

FIGURE A1 BRIDGE RAILING CONCEPTS - CONFIGURATIONS AND LOADING PATTERNS

TABLE A1 Bridge Railing Design Information

	Quantity Designations	Bridge Railing Loads and Load Distribution and Location — Railing Performance Level: PL-1	PL-2	PL-3	Optimal PL-4	Optimal PL-4T
Group I* Loads (Body and Wheels)	F_{BWH}	30 Kips	80 Kips	140 Kips	200 Kips	200 Kips
	F_{BWL}	±9 Kips	±24 Kips	±42 Kips	±60 Kips	±60 Kips
	F_{BWV}	+12 Kips (down) −4 Kips (up)	15 Kips (down) −5 Kips (up)	+18 Kips (down) −6 Kips (up)	+18 Kips (down) −6 Kips (up)	+18 Kips (down) −6 Kips (up)
Group II* Loads (Trailer Floor)	F_{FH}	—	—	—	240 Kips	200 Kips
	F_{FL}	—	—	—	±60 Kips	±50 Kips
	F_{FV}	—	—	—	−18 Kips (down) −6 Kips (up)	+18 Kips (down) −6 Kips (up)
Group III* Loads (Tank Trailer)	F_{TH}	—	—	—	—	200 Kips
	F_{TL}	—	—	—	—	±50 Kips
	F_{TV}	—	—	—	—	+18 Kips (down) −6 Kips (up)
Load Distribution Pattern Dimensions	a	24"	28"	32"	36"	36"
	b	12"	14"	16"	18"	18"
	c	—	—	—	12"	12"
	d	—	—	—	6"	6"
	e	—	—	—	—	36"
	f	—	—	—	—	8"
Load Locations	h_{BW}	16" thru (H-6")	17" thru (H-7")	18" thru (H-8")	19" thru (H_{BW}-9")	19" thru (H_{BW}-9")
	h_F	—	—	—	51"	51"
	h_T	—	—	—	—	74" (min) 84" (max)
Railing Geometry Dimensions	H	27" (min)	32" (min)	42" (min)	54" (min)	78" (min)
	H_A	10" (max)	10" (max)	10" (max)	10" (max)	10" (max)
	H_{BW}	27" (min)	32" (min)	42" (min)	32" to 42"	32" to 42"
	H_{BWR}	12" (min)	12" (min)	12" (min)	12" (min)	12" (min)
	H_F	—	—	—	54" (min)	54" (min)
	H_{FR}	—	—	—	6" (min)	6" (min)
	H_T	—	—	—	—	78" (min)
	H_{TR}	—	—	—	—	8" (min)

See Figure A1 for location of dimensions and forces.
* Each set of Group Loads to be applied separately.

RAILING LOADS

Table A1 and Figure A1 combine to give magnitude, distribution, and vertical locations of railing design loads. The horizontal loads (F_{BWH}, F_{FH}, and F_{TH}) shown in Figure A1 are assumed to be uniformly distributed over the load areas (ab, cd, and ef, respectively). The loads within each of the loading groups in Table A1 are to be applied simultaneously. However, only one group of loadings is to be applied at a time. Where a load area bears on a single longitudinal rail element that has a vertical dimension less than the related load area, the entire load should be assumed to be uniformly distributed over the horizontal projection of the rail element on the related load area. Where a load area bears on more than one longitudinal rail element, the load to be assigned to an individual element is the total load on the load area times the ratio of the vertical dimension of the horizontal projection of the rail element on the load area to the sum of the vertical dimensions of all the horizontal projections of rail elements intersecting the load area.

Note that Table A1 indicates that load F_{BWH} is to be applied over a range of heights. Loads on posts are assumed to be transmitted through the longitudinal rail elements. Longitudinal and vertical loads are to be applied at the traffic faces of the longitudinal rail elements. Longitudinal loads are to be distributed to no more than three posts.

The loading patterns and values presented in Figure A1 and Table A1 are based on estimates of forces required to effectively resist the initial impact by a tracking, single-unit design vehicle or, in the case of a tractor trailer combination, the initial impact by the tractor and subsequently by the trailer and the rear wheels of the tractor. For large single-unit vehicles such as buses there may be a "tail-slap" force that is much higher than the indicated design forces. However, this force will be distributed over a considerable length of railing. The assumption is made that a railing designed to carry the relatively concentrated loads presented in Figure A1 and Table A1 will perform adequately even though the total tail-slap loads, which will be distributed over a considerable length of railing, may be much higher than the loads given in the table. This assumption is based on an assumption that the post spacing in beam-and-post railings will be short enough to ensure several posts will resist the tail-slap. The 10-foot maximum post spacing requirement suggested in the section on railing geometry is intended to ensure this.

ANALYSIS

The performance level selection procedures in the guide specifications are premised upon an assumption that a railing will be near its ultimate strength when subjected to the maximum containment condition anticipated under the railing's nominal performance level. Thus, for economy, balance, and consistency of design, railing analysis and design should, desirably, be based on ultimate strength approaches. Advanced texts on structural design provide guidance on these approaches. Dr. T.J. Hirsch of the Texas Transportation Institute, in an August 1978 report titled "Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks," demonstrated two such approaches through the application of plastic analysis to metal bridge railings and yield-line theory to reinforced concrete railings. (Dr. Hirsch's report numbered FHWATX78-230-2 is available from the National Technical Information Service, Springfield, Virginia 22161.)

CONTINUITY AND DECK DESIGN

Within reasonable economic limits, a bridge railing should provide moment, shear, and tensile continuity throughout its length. Providing these features will require designing end anchorages and continuity transfer splices and expansion devices for rails of beam-and-post railings and continuity transfer devices for open joints in concrete parapets. It is suggested that continuity devices provide the full moment and shear capacity of the elements they connect and, for metal rails, at least 50 percent of the tensile strength of the gross cross section of the rail.

For maintenance purposes, it is highly desirable that bridge decks, railing attachments to bridge decks, and bridge railings be designed so that impact damage to a railing does not carry into the bridge deck or other bridge elements. Special attention should be given to the forces deck mounted posts can transmit to a deck and to cast-in anchor bolts. Posts transmit large concentrated loads near the edge of the deck, making it essential to check all the potential bending and shear failure modes a post loaded to its ultimate strength can induce in a deck. Similarly, it is essential to check to see that the bond development length requirements for the reinforcing steel are met.

APPENDIX B

APPENDIX B

DEVELOPMENT OF PERFORMANCE LEVELS AND PERFORMANCE LEVEL SELECTION PROCEDURES FOR BRIDGE RAILINGS

INTRODUCTION

Benefit-cost analysis and engineering judgement were combined to produce the performance levels and selection procedures presented in the guide specifications to which this appendix is attached. In the early development efforts many more performance levels were anticipated than the three finally included in the guide specifications. In fact, in the early efforts a nearly infinite number of performance levels was used for analysis purposes and then several discrete performance levels selected on the basis of what were thought to be rather sizable steps in railing strength (performance requirements) and estimated numbers of sites where a performance level would be applicable. While this approach had the advantage of theoretically better matching railing designs and costs to needs, and came close to assuring that most reasonably designed railings would fit well in some performance level niche, it presented practical problems from the standpoint of railing design development and qualification and in managing the application of many performance levels. Thus, the decision was made to recommend only three performance levels, with the understanding that there may be sites where consideration should be given to railings superior to those included in the guide specifications.

Engineering judgement was also brought to bear on the question of what the performance levels should be. There was a very strong consensus among the bridge engineers of the country that well designed railings meeting the requirements of the 1989 AASHTO *Standard Specifications for Highway Bridges,* and of earlier editions of those specifications with similar requirements, were suitable for most rural arterial highways. The railing most often cited in this regard was the New Jersey concrete parapet. Thus, there was a conscious effort made to match the middle performance level (PL-2) to the performance limit of the New Jersey concrete parapet. The lowest performance level (PL-1) was set after considering both the conditions under which it would be applicable and the practical limitations on further reduction in the cost of a realistically designed railing. The top performance level (PL-3) requirements given in the guide specifications were chosen, in part, because some highway agencies looking for traffic barriers with performance levels beyond those assured by the 1989 specifications had employed barriers with performance characteristics similar to those expected from guide-specifications PL-3 railings. Also, analysis suggested that PL-3 railings would be appropriate for a significant number of sites and would give acceptable service for all but the most extraordinary of sites. Additionally, its requirements are likely to be met by railings with heights that are visually acceptable to the public.

Once the railing performance levels were established, they were used as input to a computer-based benefit-cost analysis program (BCAP) to develop the railing performance level selection tables presented in the guide specifications. The following is a brief discussion of that program and how it was used to develop the selection tables.

BCAP

BCAP is a general purpose roadside feature analysis program. For a given set of roadway and roadside conditions it estimates roadside encroachments, the consequences of those encroachments, and the cost of the consequences and, when more than one set of conditions is input, compares the relative benefits and costs associated with each set of conditions to produce incremental benefit-cost ratios showing the relative merits of instituting (or maintaining) one set of conditions as opposed to another.

ENCROACHMENT FREQUENCY

BCAP will accept a user supplied encroachment frequency. However, it contains a default value of 0.0005 encroachments (to one side of road)/mile/year/vpd, which is what was used in developing the guide specifications. This value was selected as the most likely after a study of several conflicting data sources. Note that this rate applies to one roadway edge and to traffic in one direction. Thus, for an undivided two-way highway, the total encroachment rate to one side of the roadway will be greater by the contribution from opposing traffic. Figures B1, B2,

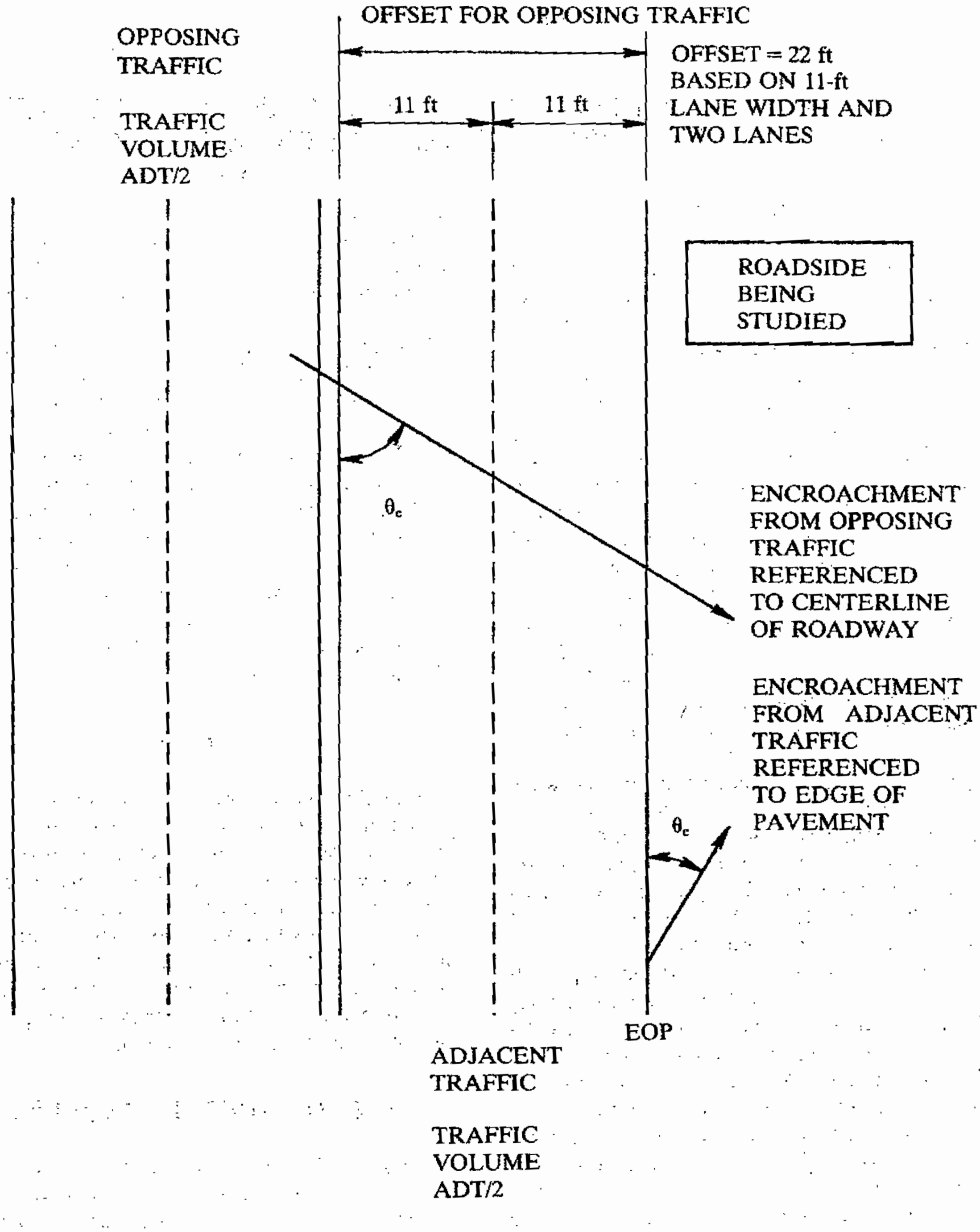


FIGURE B1 Four-lane, two-way, undivided roadway.

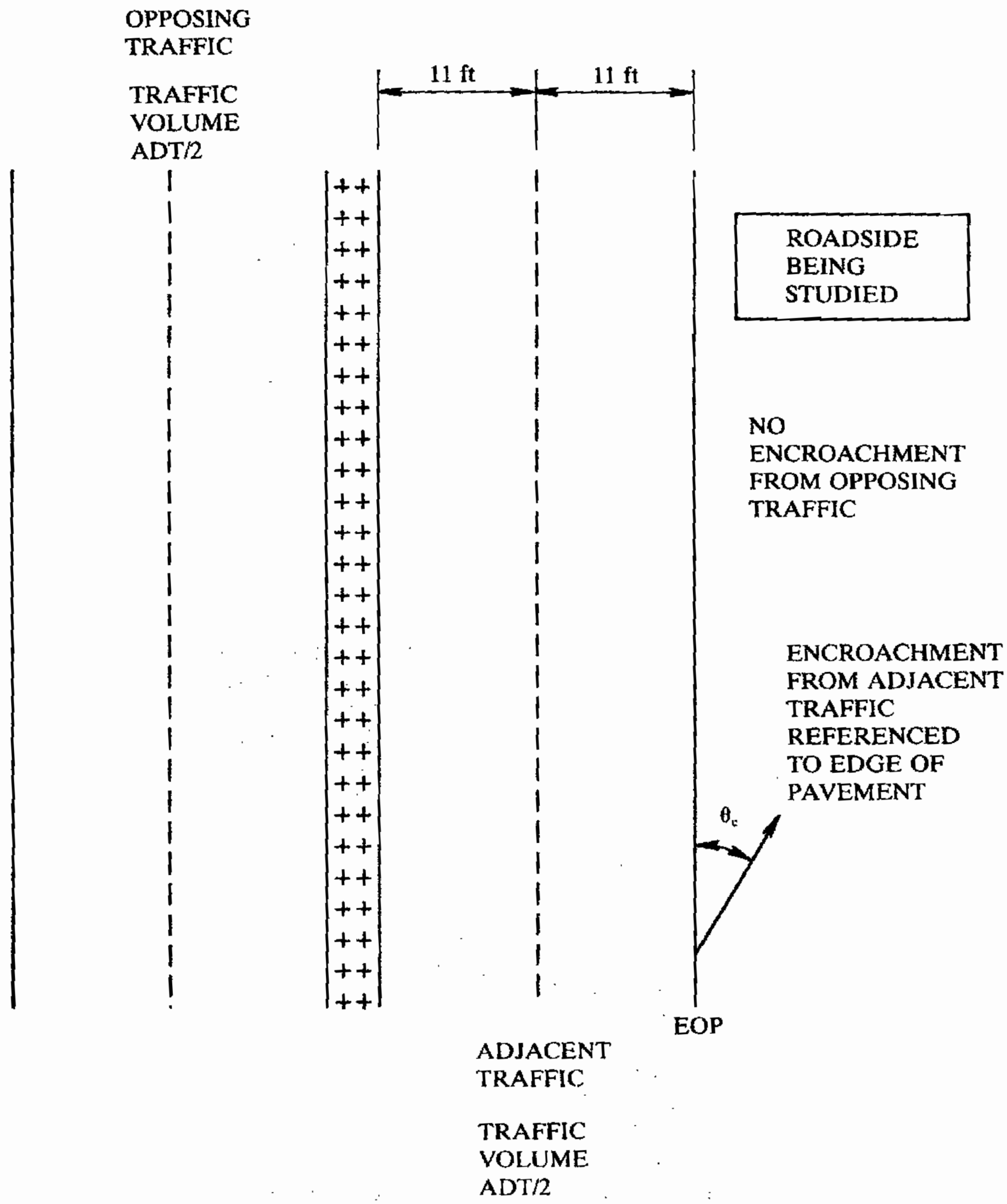


FIGURE B2 Four-lane, two-way, divided highway.

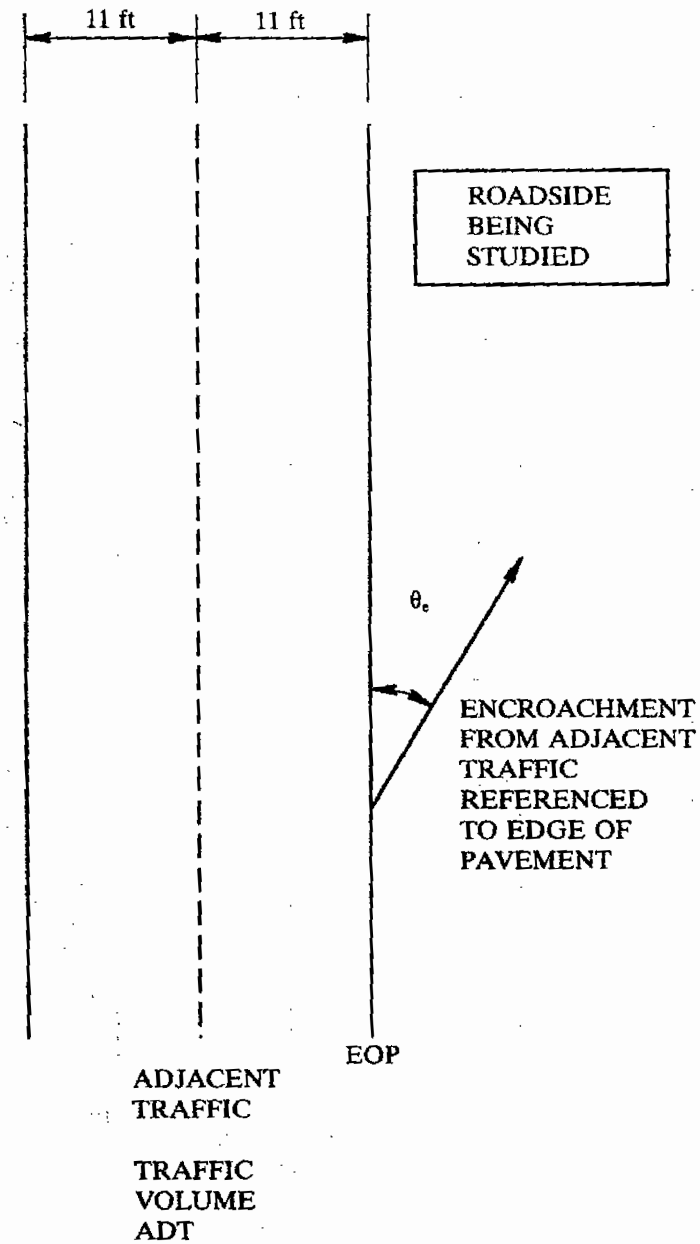


FIGURE B3 Two-lane, one-way, roadway.

and B3, respectively, illustrate encroachments for two-way undivided, two-way divided, and one-way highways. (In both BCAP and the guide specifications, the user enters with construction year ADT for the entire highway and then identifies the type of highway.)

ENCROACHMENT FREQUENCY ADJUSTMENTS

BCAP adjusts the encroachment frequency for the grade and curvature of the highway. Figure B4 illustrates these adjustment factors. In the guide

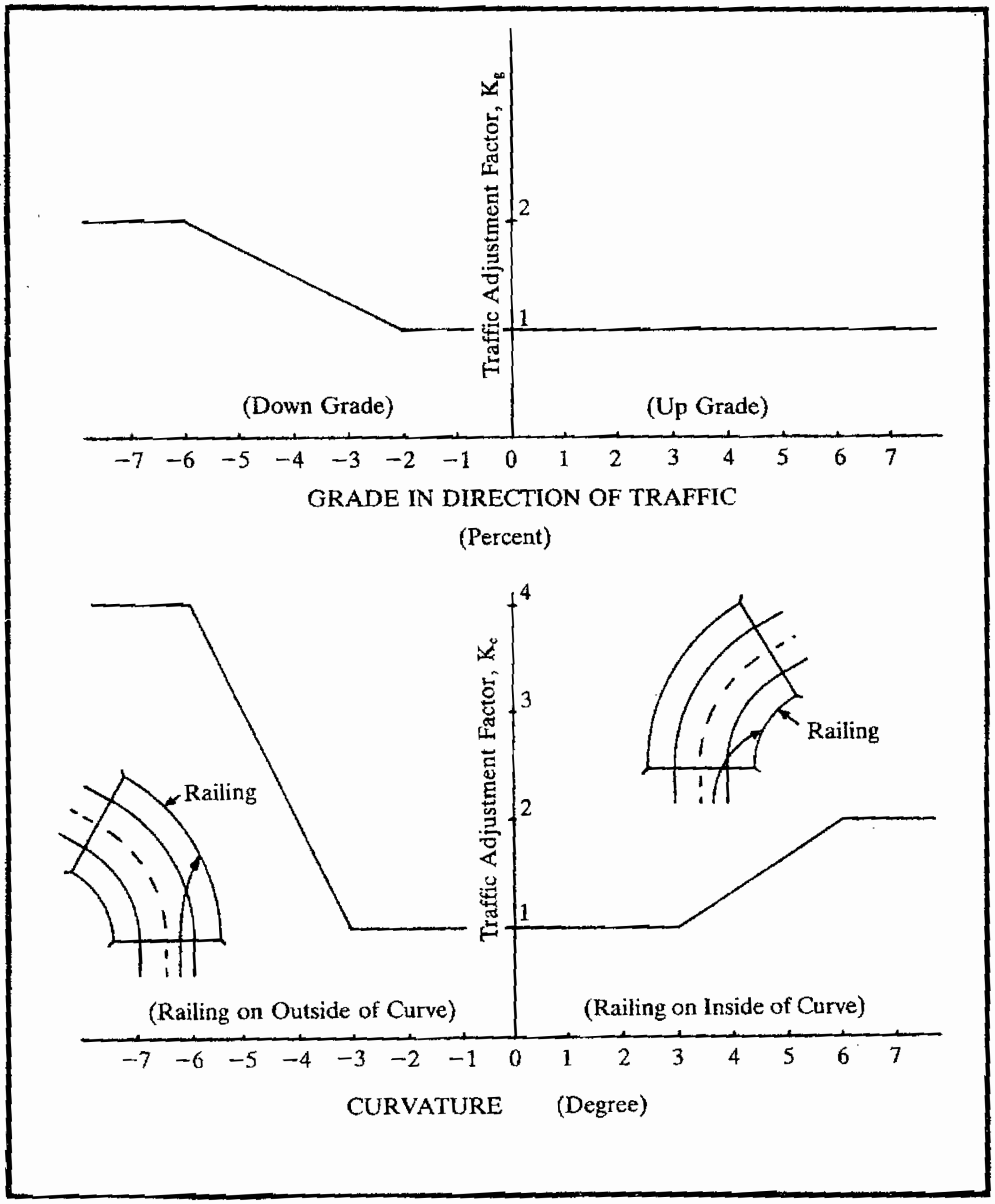


FIGURE B4 Encroachment adjustment factors.

specifications, the user applies these adjustments to the ADT before entering the performance level selection tables.

In order to account for the effect of traffic growth and to allow for the comparison of different life roadside treatments, BCAP has the user input the construction-year traffic volume (ADT), project life, and annual traffic growth rate. A 2% default growth rate is in the program. This value was used in preparing the selection tables in the guide specifications, along with a 30-year project life.

NOTE: Construction-year ADT is the estimated traffic when a project is opened, assuming this estimate includes the contribution from any soon to be opened highways or traffic generators. To convert design-year ADT to construction-year ADT for input into BCAP use the following formula.

$$\text{Const. year ADT} = \frac{\text{Des. yr. ADT}}{(1 + \text{growth rate})^n}$$

where n = number of years between construction year and design year.

If it is necessary to calculate the construction-year ADT for use in the performance level selection procedure in the guide specifications or if the 2% growth rate implicit in the tables is not acceptable, a 30-year after the construction-year ADT should be estimated using the growth rate considered appropriate for the site and this value divided by 1.81 (which is 1.02 to the 30th power) to obtain the construction-year ADT. (Any error introduced in the selection procedure through this approximation is considered to be within the accuracy of the procedure.)

THE ENCROACHMENT

The encroachment model investigates encroachments by 13 vehicle types, each leaving the highway at 10 speeds and at up to 12 angles for each speed. Thus, up to 1560 encroachment conditions are investigated to estimate the probable accident cost associated with a given set of highway conditions and roadside conditions. This estimated cost is the sum of the severities, in dollars, associated with all the vehicle, speed, and angle path combinations multiplied by the associated probabilities of their occurrences.

VEHICLES AND TRAFFIC MIX

The 13 vehicles are divided into four vehicle classifications—automobiles (4 sizes), pickups and vans (3 sizes), medium trucks (3 sizes), and combination trucks (3 sizes). In the order cited, Table B1 lists characteristics of these vehicles and the percentage of each vehicle size within a vehicle classification.

Early efforts to identify appropriate traffic mixes for analysis purposes focused on highway functional classifications, but functional classifications proved

TABLE B1 Vehicle Descriptions

Vehicle Types	Vehicle Number	Weight (lb)	Width (ft)	Length (ft)	CG Height (in)	Percent in Class
Automobiles	1	2,000	5.5	13.5	19.0	17
	2	2,700	6.0	15.0	20.0	31
	3	3,635	6.5	18.0	21.0	30
	4	4,500	6.5	18.0	21.5	22
Vans and Pickups	5	4,000	5.5	15.0	27.0	29
	6	5,500	6.5	16.5	30.0	39
	7	7,000	7.5	18.0	36.0	32
Single-Unit Trucks	8	8,000	7.5	18.0	43.0	34
	9	17,500	8.0	30.0	53.0	38
	10	30,000	8.0	35.0	68.0	28
Combination Trucks	11	30,000	8.0	55.0	52.0	33
	12	50,000	8.0	55.0	63.0	33
	13	75,000	8.0	55.0	78.0	34

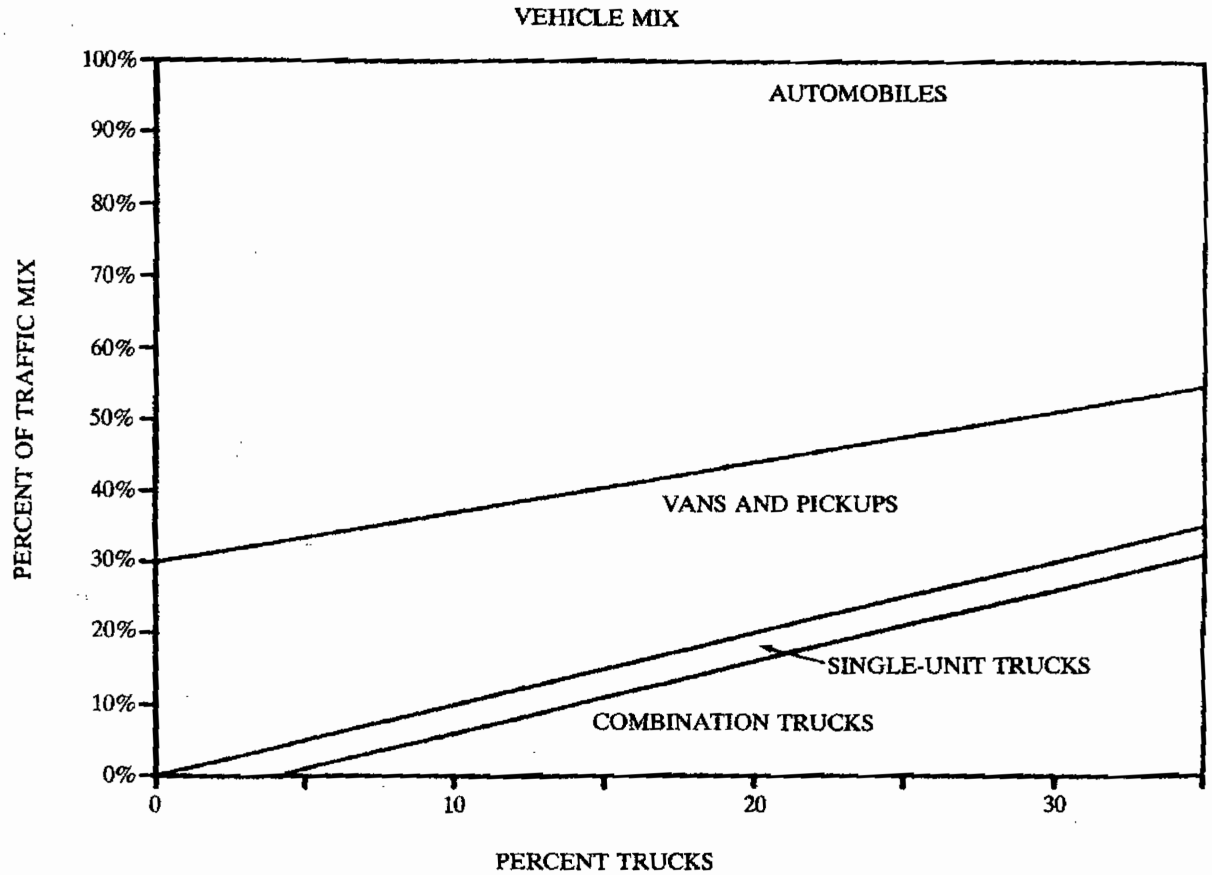


FIGURE B5 Vehicle mix based on percent trucks.

to be unreliable predictors of traffic mix. However, wh[illegible] large number of observed mixes were arrayed on the basis of the nominal percent trucks (medium plus heavy trucks), which is the information most highway designers will have regarding the vehicle mix for a highway, a very strong correlation with the other vehicle classifications appeared. Figure B5 shows the relationships adopted for BCAP. The division between automobiles and vans and pickups was the least certain in the data studied. However, because of the similarity between the two vehicle classifications, the effect of any errors in distribution will probably be small.

ENCROACHMENT SPEEDS

Vehicles can and do leave the traveled way at a variety of speeds. All the available data on encroachment speed and angle combinations come from reported accidents, which represent a very small portion of total encroachments, and the information on the reported encroachments is often unreliable. A decision was made to develop rational speed-angle relationships for use in BCAP. When the effects of the postulated relationships were tested against the limited available data, the correlation was very good.

Figure B6 shows the basic assumptions made regarding encroachment speed. The reference speed (RS) shown in the figure is assumed equal to 0.9 times the highway design speed. It was further assumed that the maximum encroachment speed would be 15 mph greater than the reference speed, that the probability of encroaching at any speed from 5 mph below to 5 mph above the reference speed would be the same, and that the probability of any encroachment speed above or below the 10 mph band centered at RS would decrease linearly as the maximum speed was approached on the high side and as 0 mph was approached on the low side. To obtain discrete

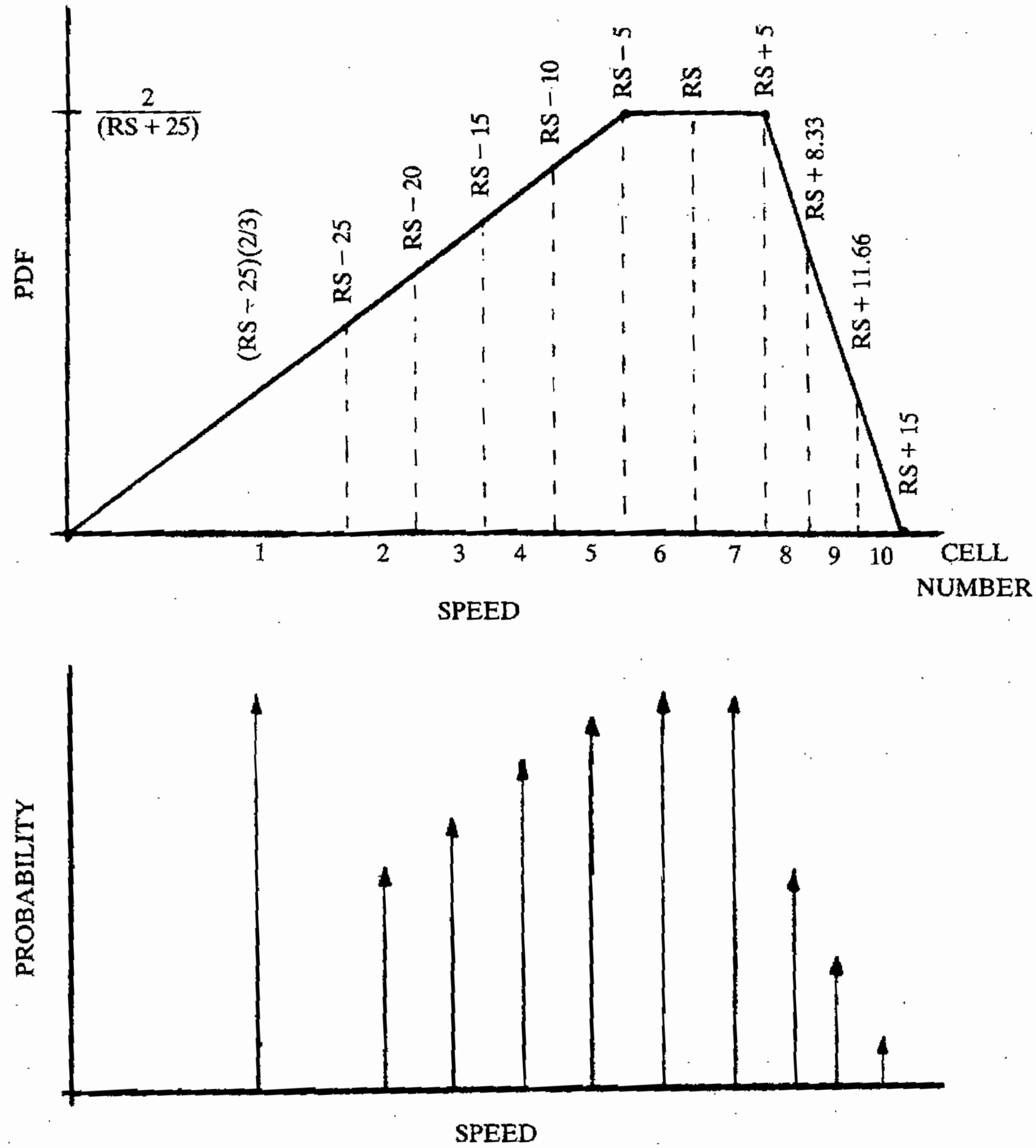


FIGURE B6 Probability density function for encroachment speed.

probabilities and speeds to use in BCAP, the probability density function (PDF) curve shown in the top half of Figure B6 was divided into 10 cells as shown. (Note that, because of the cell definitions used, RS can not be less than 25 mph, or the design speed less than 27.8 mph, without cell 1 being associated with a negative speed. Therefore, BCAP should never be used with a design speed below 27.8 mph.)

The area under the probability density curve is 1 and the area of a cell is the probability an encroachment will be at a speed within the limits of the cell. In BCAP the probability of an encroachment occurring at a speed within the limits of a cell is assigned to the central speed of the cell for every cell except cell 1, where the probability is assigned to a speed equal to 2/3 the width of the cell.

ENCROACHMENT ANGLE

For a vehicle starting from going straight ahead on a tangent traveled way, the maximum angle it can leave the traveled way without skidding or upsetting is dependent upon the offset of the vehicle from the edge of the traveled way, the speed of the vehicle, the coefficient of friction between its tires and the pavement, the stability of the vehicle, and the minimum turning radius of the vehicle. In BCAP the effect of a vehicle's turning radius is not checked because it is assumed it would only come into play at low speeds and smaller offsets than are likely to be relevant. On the other hand, while vehicle stability is not a factor in controlling the maximum exit angle for automobiles, it can be a significant factor for high-center-of-gravity trucks. Table B2 shows the friction coefficients associated with static stabilities of each BCAP vehicle based on its width and center of gravity height. On the advice of experts in truck handling, values significantly less than the static stability values were used in determining the departure angles used in developing the selection table in the guide specifications. Note that, since the selection tables were developed, small changes have been made in the coefficients in BCAP. The values currently in BCAP, and those used to develop the selection table, are both shown in Table B2. The difference between the two sets of available friction coefficients is small, with the set used in developing the selection table giving slightly more conservative results. In developing the coefficients it was assumed the coefficient of friction between tires and pavement could not exceed 0.80 and, as stated before, expert advice was used in setting coefficients for the large vehicles. (After the bridge railing selection tables were completed it was observed that the values used for the larger vehicles were approximately their static stability coefficients less 0.30. Because this rule reduced anomalies seen in the predicted behavior of some vehicles, it was adopted for use in BCAP. However, the change was not considered to have sufficient effect to require recalculating the values in the selection table.)

The way the available coefficient of friction is utilized in BCAP is as follows:

- On the basis of limited available data, maximum encroachment angle is set at 36 degrees.
- For each vehicle, the probability of an encroachment occurring is assumed to be greatest at 0 degrees and decreases linearly to zero at 36 degrees, as shown in the upper half of Figure B7, unless the vehicle, because of its speed, offset, and available friction coefficient, can not achieve a 36-degree encroachment

TABLE B2
Friction Coefficients for Each Vehicle

				Maximum Available Friction	
Vehicle Number	**Weight (lb)**	**Type of Vehicle**	**Static Stability**	**Bridge Rail Study**	**Present BCAP Model**
1	2,000	Passenger Car	1.74	0.8	0.80
2	2,750	Passenger Car	1.80	0.8	0.80
3	3,635	Passenger Car	1.86	0.8	0.80
4	4,500	Passenger Car	1.81	0.8	0.80
5	4,000	Light SU Truck	1.22	0.8	0.80
6	5,500	Light SU Truck	1.30	0.8	0.80
7	7,000	Light SU Truck	1.25	0.8	0.80
8	8,000	Heavy SU Truck	1.05	0.7	0.75
9	17,500	Heavy SU Truck	0.91	0.6	0.61
10	30,000	Heavy SU Truck	0.71	0.5	0.41
11	30,000	Combination Truck	0.92	0.6	0.62
12	50,000	Combination Truck	0.76	0.5	0.46
13	75,000	Combination Truck	0.62	0.4	0.32

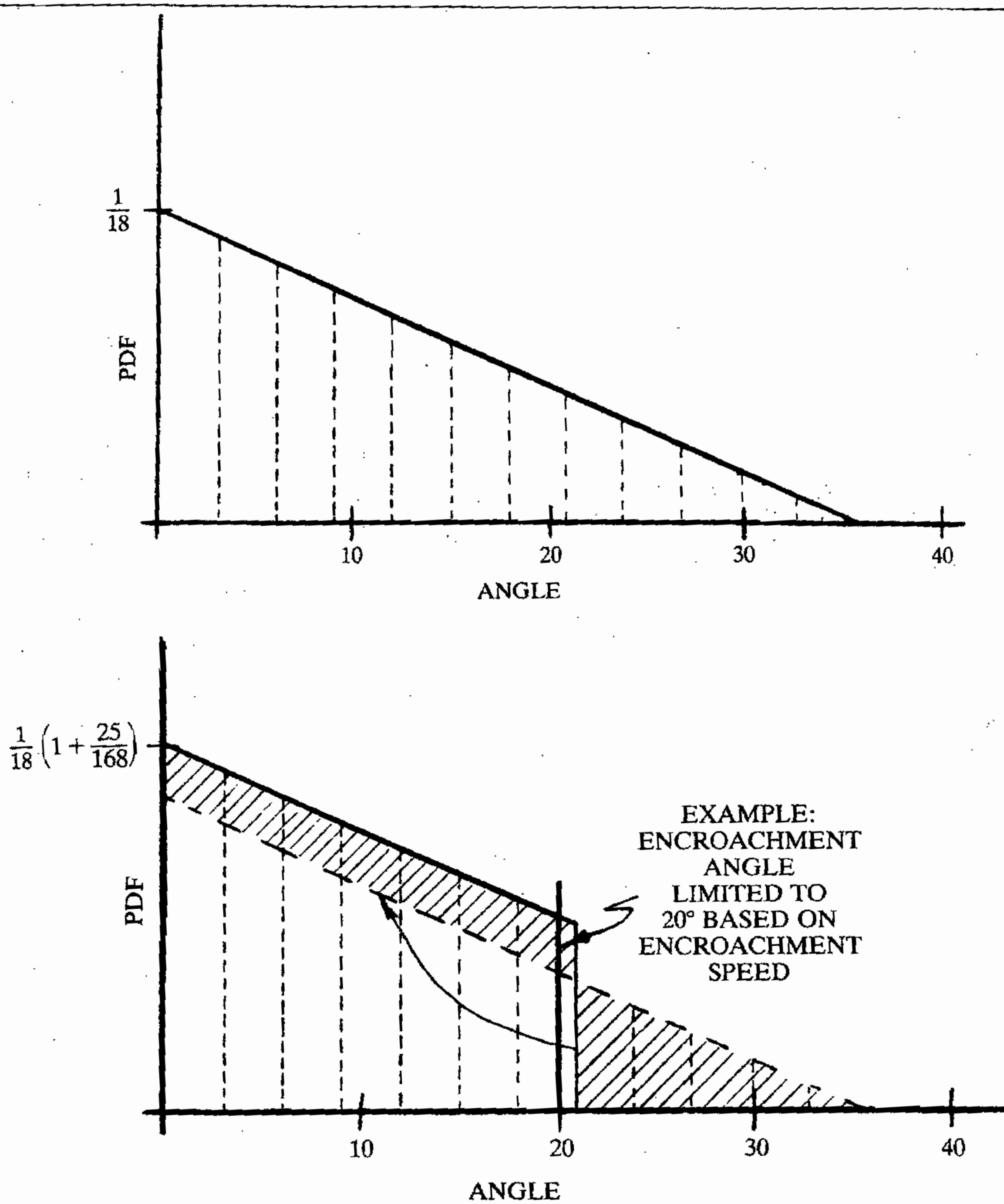
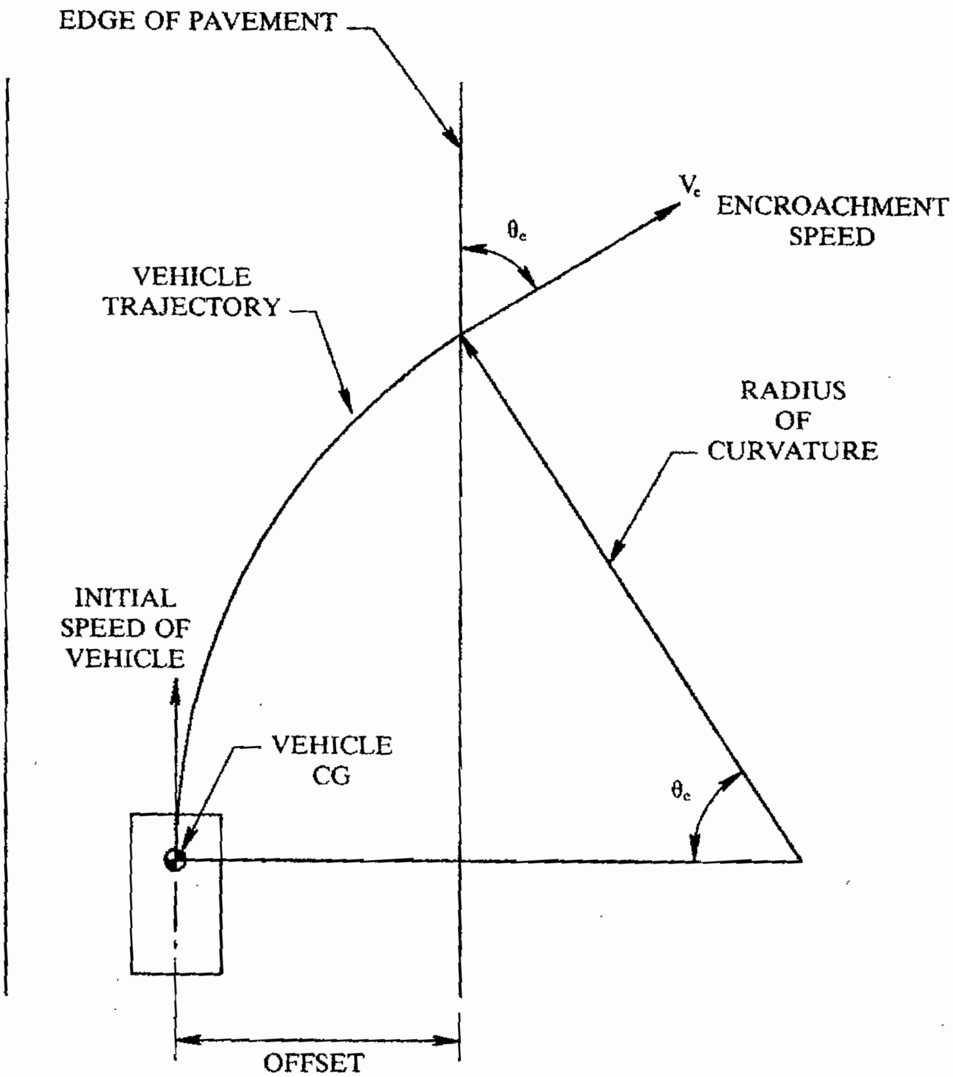


FIGURE B7 PDF for encroachment angle.

angle, in which case the probability density function curve for the vehicle is adjusted. (Figure B8 shows the method for calculating the maximum encroachment angle. The offset used in the selection table development was 18 feet.)

- The 36-degree wide probability function curve is divided into 12 three-degree-wide cells. The central angle of a cell is the angle used in estimating the consequence of encroachments at angles within the cell. The probability of occur-



$R_e = V_e^2/gf_m = S_o/(1 - \cos\theta_e)$
$\theta_e = \cos^{-1}(1 - S_o g f_m/V_e^2)$
= Maximum encroachment angle
V_e = Encroachment speed = Initial speed of vehicle
S_o = Vehicle offset from edge of pavement
f_m = Maximum available friction coefficient
R_c = Minimum radius of curvature
g = Acceleration of gravity

FIGURE B8 Encroachment angle model.

rence assigned to a central encroachment angle is the probability of all the angles within its cell, which is equal to the area of the cell.

- When the maximum achievable angle, based on vehicle and speed, is less than 33-degrees, BCAP searches for which of the three-degree cells below the 12th cell the maximum achievable angle falls and the probability density function curve is adjusted by eliminating all the higher angle cells and distributing their area to the remaining cells as illustrated in the example in the lower half of Figure B7. The slope of the adjusted PDF curve is assumed to be the same as that of the unadjusted curve and, since the area added above the line of the unadjusted curve equals the area eliminated to the right of the cell within which the maximum estimated encroachment angle falls, the area of the adjusted PDF curve remains 1. Note that when the maximum encroachment angle for a vehicle is limited because of the encroachment speed, the number of encroachment angles investigated in BCAP for the vehicle at the limiting speed is reduced by the number of cells eliminated.

ENCROACHMENT TRAJECTORY

For each of the 13 vehicles at each of the 10 encroachment speeds at each of the achievable encroachment angles (12 if not limited by encroachment speed), BCAP estimates an encroachment trajectory. The assumption is made that the maximum potential lateral extent of an encroachment, beginning with a given encroachment speed and angle, will be defined by a vehicle traveling in a straight line and subjected to a constant deceleration beginning when the vehicle leaves the traveled way (crosses the edge of pavement). This is illustrated in Figure B9. In developing the selection tables in the guide specifications a deceleration rate of 13 ft/sec^2 was used, which is the equivalent of a braking coefficient of friction of 0.404.

Obviously, the trajectory used to set the maximum lateral extent of encroachment is just one of a limitless number of trajectories a vehicle might take after leaving the traveled way at a given speed and angle. To account for vehicles that will not reach the estimated maximum encroachment limit because of steering and braking differing from that used to estimate the maximum encroachment limit, an assumption is made that the probability of a vehicle's going laterally further from the edge of pavement than a given distance (Y_d) on the straight line trajectory is given by the formula:

$$P(Y > Y_d) = .5 + .5 \cos\pi \frac{(Y_d)}{(Y_d)} \text{ rad.} \qquad \text{for } Y_d < Y_m$$
$$= 0 \qquad \text{for } Y_d > Y_m$$

Where: Y = lateral extent of encroachment
Y_m = maximum estimated lateral extent of encroachment
Y_d = lateral distance from edge of pavement.
π rad. = 180°

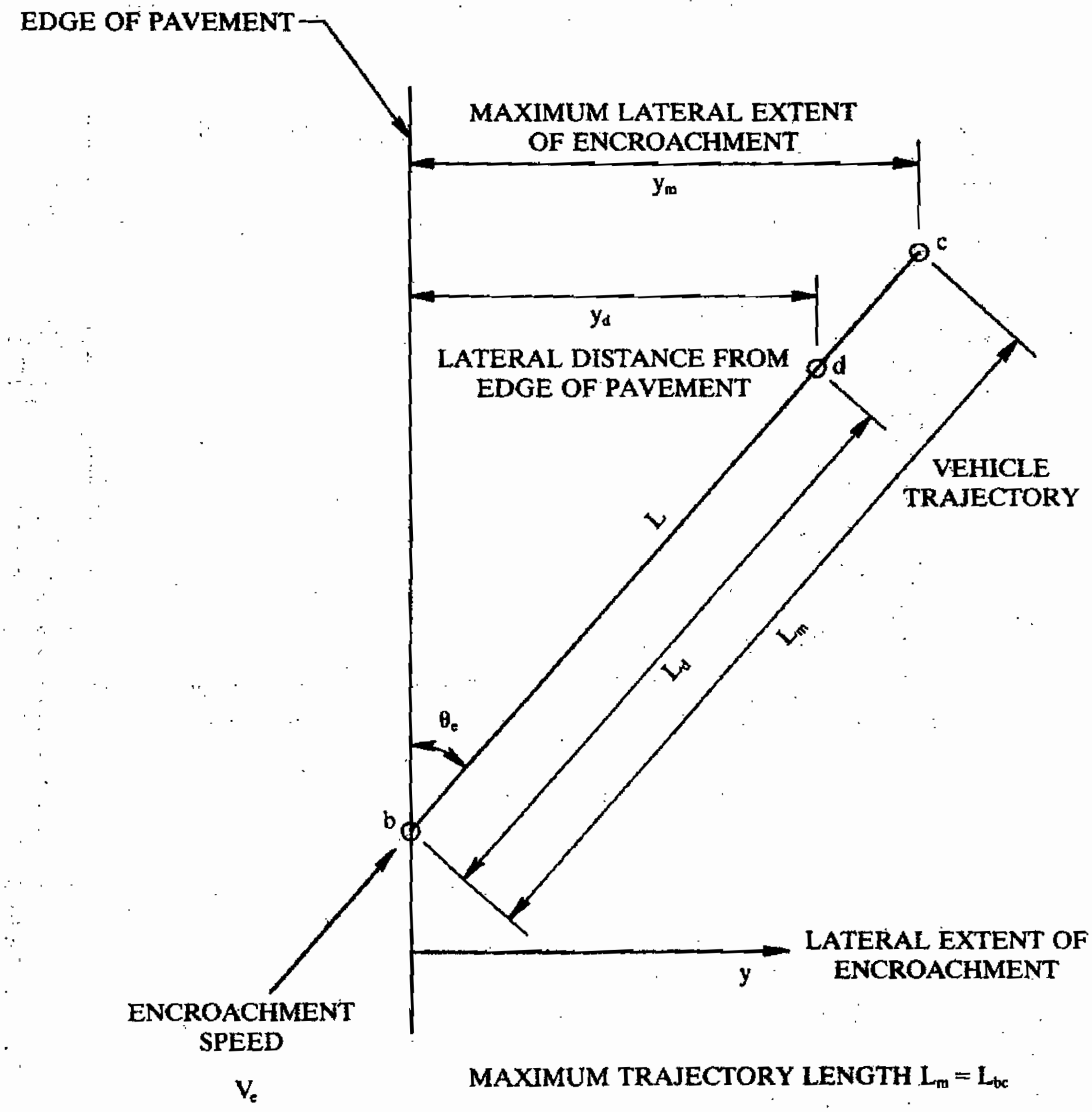
In BCAP, an encroaching vehicle is treated as maintaining its initial encroachment angle throughout its trajectory. On the other hand its speed is assumed to diminish under the influence of the same deceleration rate used to estimate the maximum extent of encroachment.

Figure B10 shows the estimated lateral extent of encroachments for highways of various design speeds carrying 20% trucks. The curves in Figure B10 are based on the assumptions in BCAP, which, except for the revisions in available friction cited in the discussion on encroachment angle, are the same as those used in developing the selection tables in the guide specifications. The curves compare favorably with the limited field data available.

Figure B11 shows the estimated maximum and average speeds for encroaching vehicles at lateral extents of encroachment for a highway with a 60 mph design speed. Figure B12 shows the estimated average vehicle speeds at lateral extents of encroachment for various highway design speeds. Limited field data suggest that these estimated average values are reasonable. (Note that these curves would be smoother and would have slightly different shapes if they were based on a larger number of, and thus narrower, speed and angle cells.)

ACCIDENT COSTS

From the BCAP vehicle characteristics, the BCAP encroachment model speed and encroachment angle estimates, input data on the strength and height characteristics of a bridge railing, and input on a railing's offset from the traveled way; BCAP predicts whether a vehicle will contact a given railing and, if it does, whether it will be redirected, penetrate the railing (break through or rollover the railing), or roll over on the roadway side of the railing. BCAP, using built-in crash models and input accident cost information, estimates the accident cost



MAXIMUM TRAJECTORY LENGTH $L_m = L_{bc}$

$$L_m = V_e^2/2a$$

WHERE: V_e = ENCROACHMENT SPEED
a = DECELERATION RATE

MAXIMUM LATERAL EXTENT OF ENCROACHMENT

$$y_m = L_m \sin \theta_e$$

FIGURE B9 Geometry for computing lateral extent of encroachment.

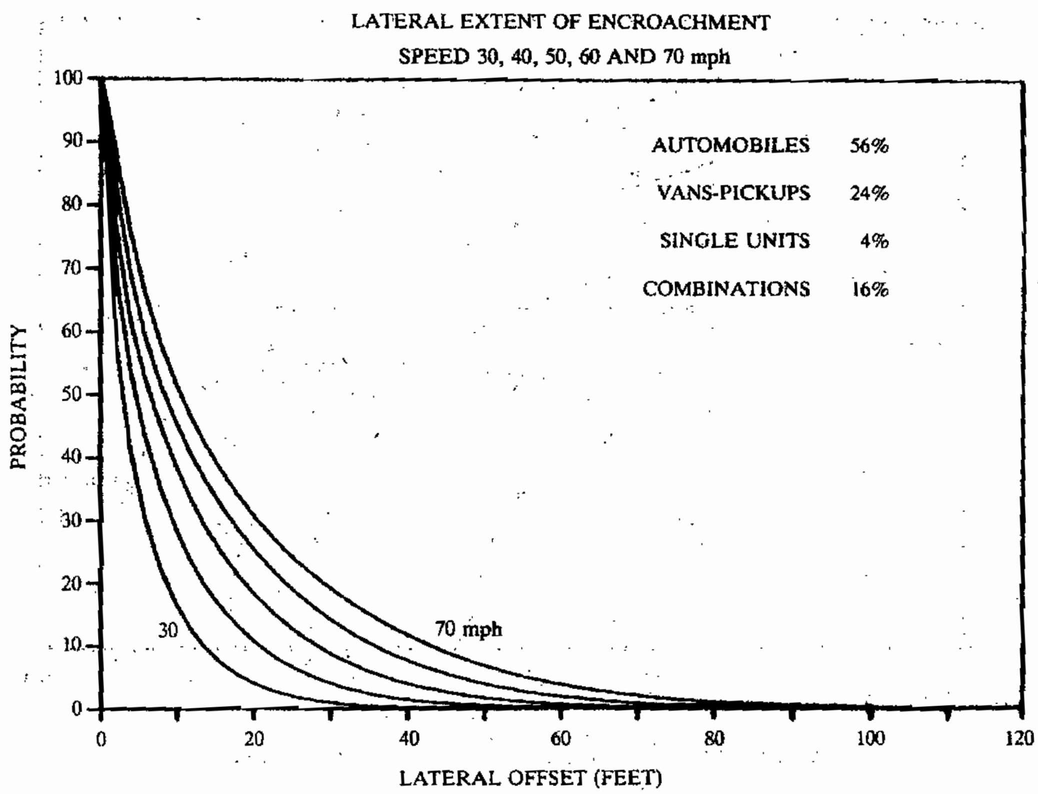


FIGURE B10 Lateral extent of encroachment.

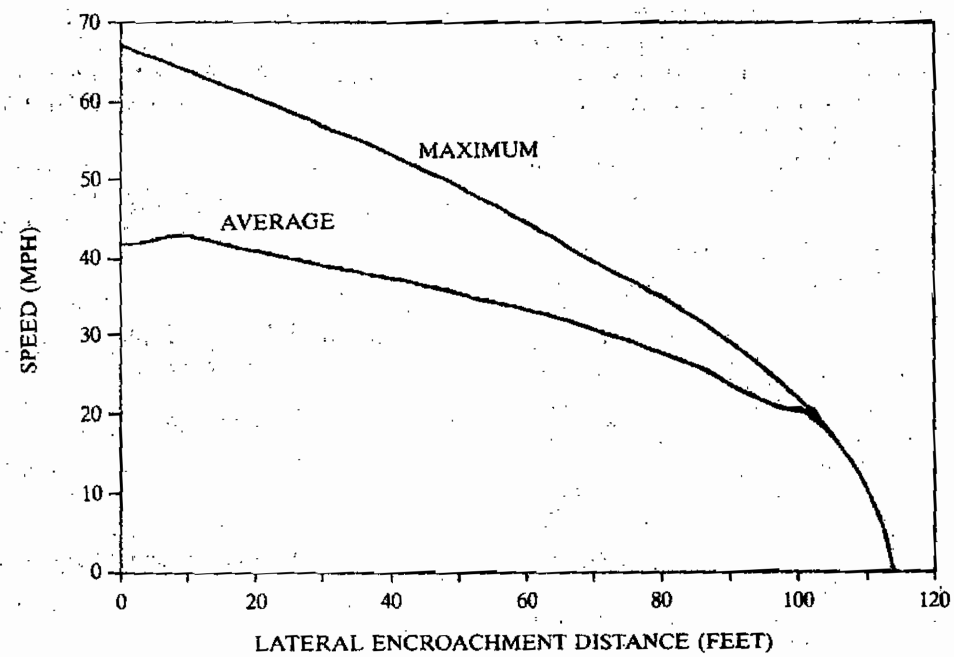


FIGURE B11 Predicted maximum and average vehicle speeds related to lateral encroachment distance (Design Speed = 60 mph)

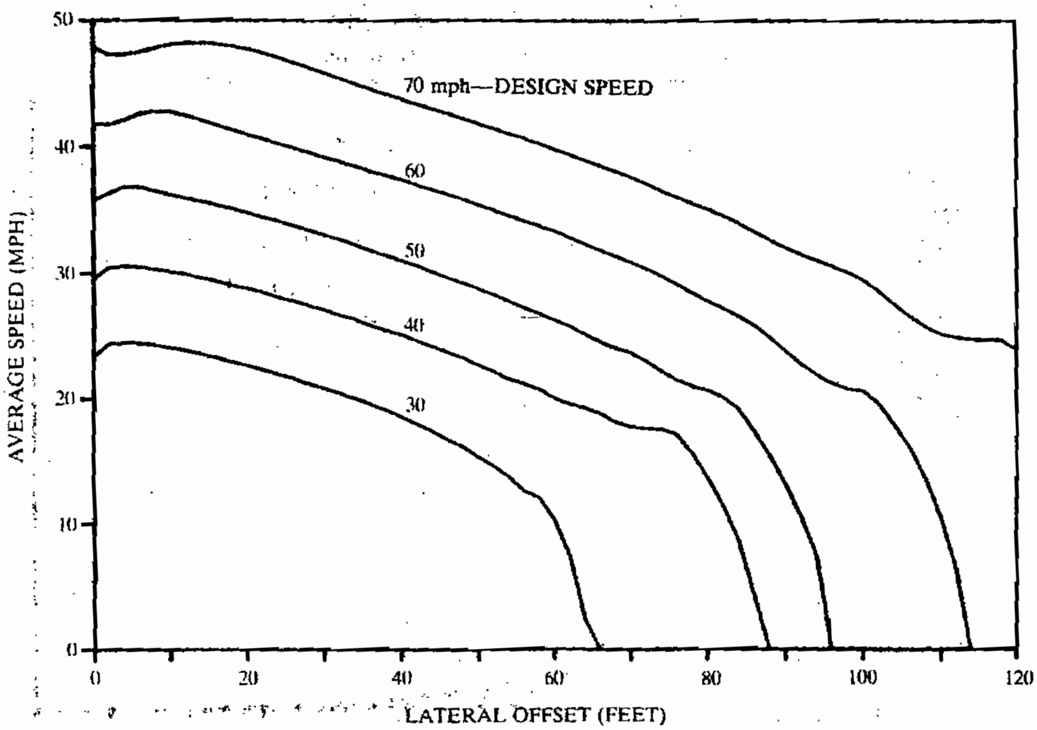


FIGURE B12 Impact speed as a function of lateral offset.

associated with each vehicle speed and angle combination assumed possible for a given design speed. Then, given the roadway type, construction-year traffic volume, percent trucks, traffic growth rate, capital recovery rate, project life, and the length and cost of the railing, BCAP, applying all the various probabilities of occurrences, can estimate the annualized and present worth accident costs associated with a specific railing.

Basic to this step is the establishment of cost-accident severity relationships. In developing the selection tables in the guide specifications, and as default values in BCAP, six accident classifications and associated costs were assumed. They are as follows:

Property Damage Only (level 1)	$500
Property Damage Only (level 2)	$2,500
Slight Personal Injury	$3,000
Moderate Personal Injury	$10,000
Severe Personal Injury	$110,000
Fatality	$500,000

These accident classifications are combined in BCAP to produce accident severity indices as illustrated in Figure B13. Table B3 shows the vehicle accident costs that result from combining the accident severity assumptions with accident classification costs. Figure B14 graphically shows the resulting severity index (SI)-cost relationship.

For the three railing contact conditions considered in developing the selection tables in the guide specifications, SI's were assumed to be the following:

- *For redirection* the SI was estimated to be linearly correlated to the lateral acceleration of the center of gravity of the vehicle as predicted by the formula:

$$G_{lat} = (V \sin \theta)^2 / 2g[(A \sin \theta) - 0.5B(1 - \cos \theta) + D]$$

Where G_{lat} = Average Acceleration (g's).
$g = 32.174$ ft/sec^2
D = Barrier Deflection (ft). Assumed to be 0 for bridge railings.
A = Distance from vehicle front end to center of mass (ft).
B = Vehicle Width (ft).
θ = Impact Angle (degrees).

The $G_{lat} - SI$ relationship was set by assuming that

$SI = 3.3$ at 4 g's
$SI = 4.5$ at 10 g's

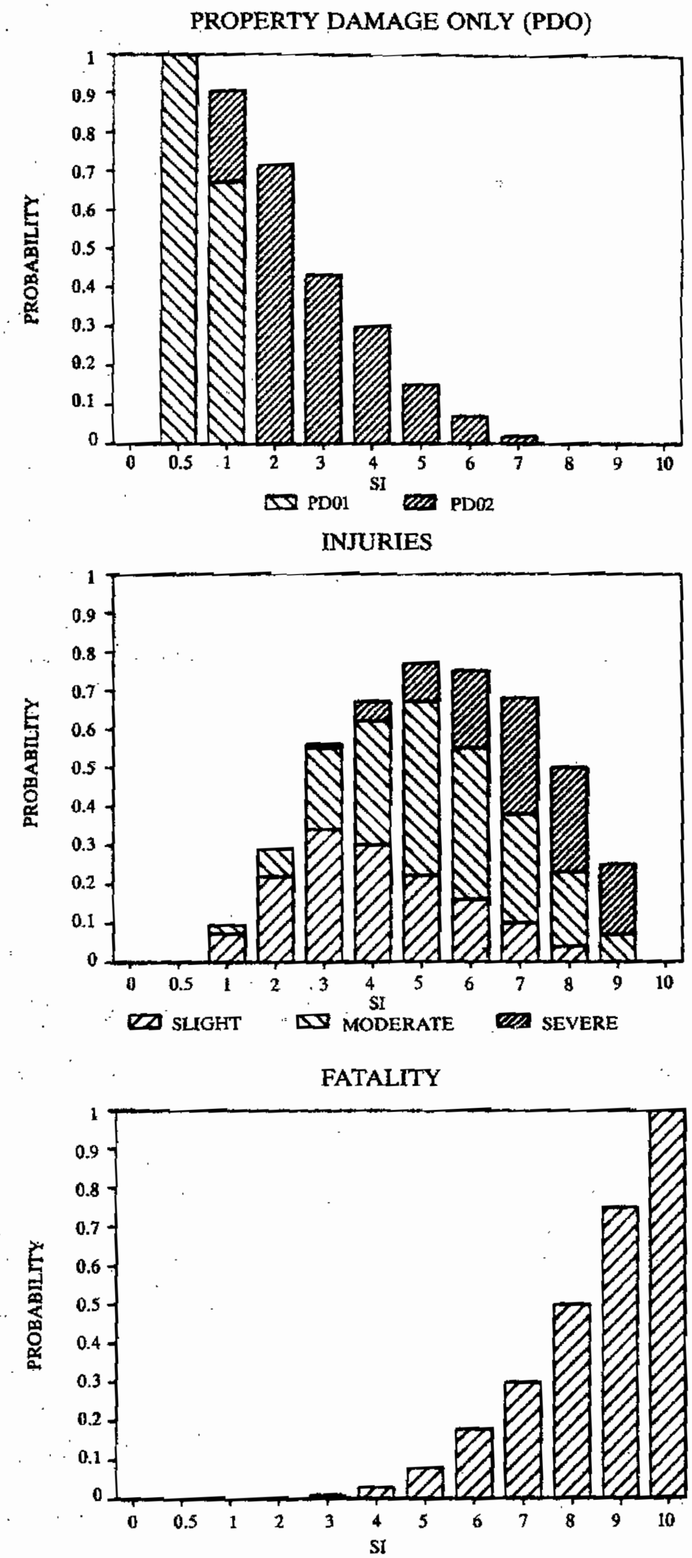


FIGURE B13 Severity indices.

TABLE B3
Severity Indices

Accident Type	Severity Index 0	0.5	1	2
No Damage	100.0%	0.0%	0.0%	0.0%
PD01	0.0%	100.0%	66.7%	0.0%
PD02	0.0%	0.0%	23.7%	71.0%
Slight Injury	0.0%	0.0%	7.3%	22.0%
Moderate Injury	0.0%	0.0%	2.3%	7.0%
Severe Injury	0.0%	0.0%	0.0%	0.0%
Fatality	0.0%	0.0%	0.0%	0.0%
Accident Cost	$0	$500	$1,375	$3,135
Accident Type	**Severity Index 3**	**4**	**5**	**6**
No Damage	0.0%	0.0%	0.0%	0.0%
PD01	0.0%	0.0%	0.0%	0.0%
PD02	43.0%	30.0%	15.0%	7.0%
Slight Injury	34.0%	30.0%	22.0%	16.0%
Moderate Injury	21.0%	32.0%	45.0%	39.0%
Severe Injury	1.0%	5.0%	10.0%	20.0%
Fatality	1.0%	3.0%	8.0%	18.0%
Accident Cost	$10,295	$25,350	$56,535	$116,555
Accident Type	**Severity Index 7**	**8**	**9**	**10**
No Damage	0.0%	0.0%	0.0%	0.0%
PD01	0.0%	0.0%	0.0%	0.0%
PD02	2.0%	0.0%	0.0%	0.0%
Slight Injury	10.0%	4.0%	0.0%	0.0%
Moderate Injury	28.0%	19.0%	7.0%	0.0%
Severe Injury	30.0%	27.0%	18.0%	0.0%
Fatality	30.0%	50.0%	75.0%	100.0%
Accident Cost	$186,150	$281,720	$395,500	$500,000

- *For penetration* the assumption was made that there would be little activity on the ground near a bridge and that the height of fall from a bridge would be 35 feet and, for that height of fall,

$$SI = 7.0.$$

Figure G2.7.1.3B in the guide specifications contains curves that can be used to estimate a traffic adjustment factor to compensate for heights of fall or under structure activities that differ from those anticipated in setting the penetration SI at 7.0.

- *For rollover* on the roadside of the railing the assumption was made that severity would be linearly correlated to the impact speed. The relationship was set by assuming

$$SI = 4.0 \text{ at } 30 \text{ mph}$$
$$SI = 5.3 \text{ at } 60 \text{ mph}$$

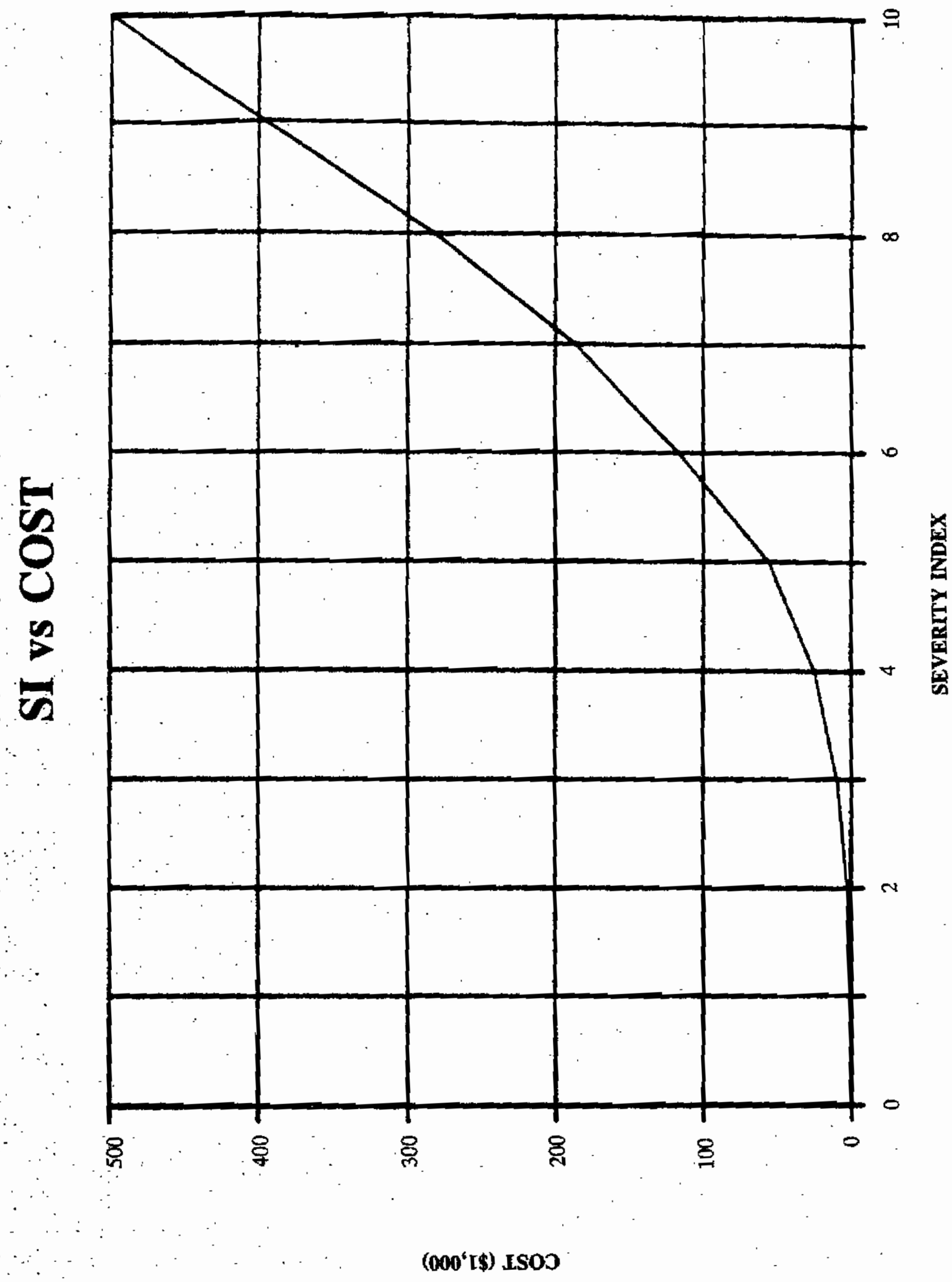


FIGURE B14 Accident cost as a function of SI.

To estimate the repair cost for a railing impact the assumption was made that if the ultimate containment strength of the railing was exceeded the cost would be the equivalent of the initial construction cost of 80 feet of railing. A railing's strength is nominally defined by its performance level. Thus, the minimum ultimate strength is that required to contain the most severe impact prescribed for the railing's performance level. In order to compare the effects different impacts might have on a railing, a vehicle impact index (I_v) equal to G_{lat} times the vehicle's weight was used. (For combination trucks only the portion of the total vehicle weight acting on the tractor is considered. See footnote 4 of the Table G2.7.1.3A of the guide specifications.) Since at some level of impact there is likely to be no railing repair cost, an assumption was made that there would be no cost for an I_v equal or less than 75 percent of the I_v associated with the containment limit of the railing. The repair cost is assumed to vary linearly from 0 at an I_v equal 0.75 of that associated with the performance limit of the railing to 80 feet at an I_v equal to the performance limit of the railing.

BENEFIT COST ANALYSIS

The railing costs per foot used in development of the selection tables are as follows:

PL-1	$28.80
PL-2	$43.62
PL-3	$68.96

Of course, actual railing costs might vary considerably from these. However, if discrete performance levels and manageable selection procedures were to be prepared, it seemed essential to establish fixed railing costs. The costs used are believed to be approximately the minimum achievable. (The apparent precision of the costs result from a railing strength-height-cost relationship prepared in order to compare several possible performance level definitions and numbers of performance levels. It is not intended to imply certainty of the estimate.)

To obtain the ADT breaks given in the selection table for a next higher performance level railing, runs were made for each combination of design speed, percent trucks, railing offset range, and railing performance level for both divided and undivided highways. The actual offset distance used to represent the various offset ranges were as follows:

Range (ft)	Distance Investigated (ft)
0–3	1
3–7	4
7–12	8
>12	12

All the runs were made using a fixed ADT to obtain annualized accident, construction, and repair costs for each set of conditions and the incremental benefit-cost ratios calculated between the various performance levels subject to the same design speed, percent trucks, railing offset, and highway type combination. These data were then processed to determine the construction-year traffic volume at which the incremental benefit-cost ratio between a railing of a given performance level and the next lower performance level became 4. This volume is the recommended threshold volume for using the given performance level railing as opposed to the next lower performance level railing. Note that the volumes given under the one-way highway type were obtained by simply dividing the volumes for the divided highway type by 2.

Figure B15 shows, for one set of highway conditions, the incremental benefit-cost ratio-traffic volume relationship for the three performance level railings defined in the guide specifications. The highway conditions are that the highway is divided, it has a 50 mph design speed, and the railing offset is in the range of 7 to 12 feet. For an undivided two lane highway, about two-thirds the traffic volume shown in the figure will produce the same incremental benefit-cost ratios. Because a decision has been made that railings will be required on all bridges, the curves comparing the three performance level railings to no railing in the figure are of interest primarily because they show that, for assumptions used to produce the selection table in the guide specification, any of the railings will pay for its self (yield a B/C ratio greater than 1 when compared to having no railing) at relatively low traffic volumes. Where the curves comparing a PL-2 railing with a PL-1 railing and a PL-3 railing with a PL-2 railing cross the incremental B/C ratio equals 4 line are where the guide-specifications-recommended breaks in performance level usage occur. As stated earlier, the B/C ratio of 4 was selected to yield a usage that fit a rather substantial consensus of what constitutes a responsible railing selection practice. One might also infer from Figure B15 that by selecting a B/C ratio of 4 as a

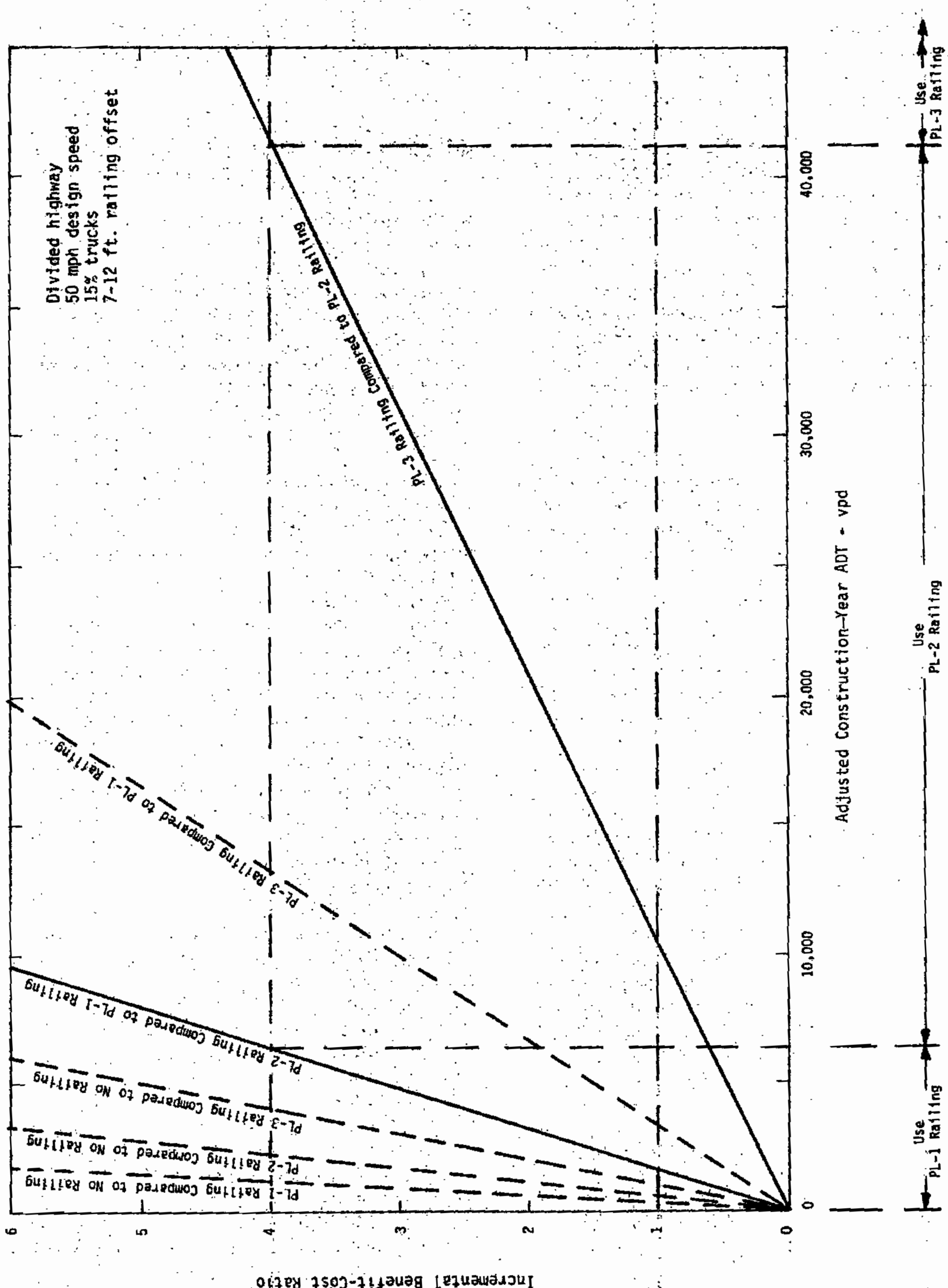


FIGURE B15 Incremental Benefit-Cost Ratios for Three Performance Level Railings for One Set of Highway Conditions break point there is not a great likelihood that any errors in the assumptions made in developing the selection table will result in wasteful expenditures on railings through use of the selection procedures.

CLOSING

The intended result of the railing performance levels and selection procedures in the guide specifications is an improved match between site needs and the performance capabilities (and costs) of railings provided. Under the guide specifications, the PL-1 railings will be superior to the railings on most existing low volume bridges and the PL-1 railings are also expected to be less expensive then railings designed to the 1989 AASHTO *Standard Specifications for Highway Bridges*. The PL-2 railing is believed to be equivalent to the better railings designed to the requirements of the 1989 specifications, which have performed well under most conditions. The PL-3 railings will fill a recognized need for railings superior to those required under the 1989 specifications. Therefore, a reasonable expectation is also that application of the guide specifications will lead to an improvement in safety, with no increase in overall railing costs and probably at a savings in overall costs.

Note: The railing requirements of the 1989 specifications are the same as those of the 1983 specifications and similar in all the *Standard Specifications for Highway Bridges* following issuance of the 1964 Interim Specifications.

COMMENTARIES

C G 2.2.5

This article recognizes that a raised sidewalk or a brush curb will have an influence on the capability of a railing to contain and redirect an errant vehicle and it requires that railings to be used with brush curbs or sidewalks be crash tested along with them. Even though crash testing of a railing with a brush curb is intended to prove the crashworthiness of the railing-brush curb combination, a maximum brush curb width limitation of 9 inches is set in the belief that that width would not have a significant adverse effect on a railing's performance under impact conditions not investigated in qualification crash tests. The desirable maximum width of 6 inches is suggested in the belief it is sufficient to produce any benefits obtainable from brush curbs and would have even less potential for adverse effect on a railing's performance than would a wider curb. Of particular concern is the potential for the brush curb to produce a ramping effect that might allow a vehicle to climb over a railing or a tripping effect that would increase the likelihood for high-bodied vehicles to roll over a railing. The latter effect comes from the low location of the force exerted by a brush curb on an impacting vehicle as the rear of the vehicle slides (slaps) into a curbed railing face.

The geometric requirements set forth for sidewalks in this article are intended to be compatible with those in the AASHTO *A Policy on Geometric Design of Highways and Streets,* which has an implied recommendation that the minimum width of sidewalk be 4 feet. This recommendation is not voided by the recognition of a 3.5-foot width sidewalk in Article G2.7.1.1.3. Instead, the intention is to establish the limits in sidewalk configurations over which the results of a series of qualification crash tests are likely to be applicable.

Coincidentally, a 3.5-foot wide sidewalk would allow two people to pass with only moderate body twisting and crowding to the outer edges of the sidewalk. Thus, such a width might be acceptable where a bridge is being rehabilitated, where there is a very low expected pedestrian traffic volume, and where providing a greater width would require an otherwise uneeded major revision of the substructure or main members of the superstructure.

C G 2.7.1.1.3

The guide specifications contain no instructions on the structural analysis of traffic railings or the traffic portions of combination railings. Instead, acceptability of a traffic railing is to be determined through crash testing. However, the expense of crash testing makes a cut-and-try approach to design impractical and precludes exhaustive testing of a railing system. Thus, guidance is needed for the structural and geometric design of a railing system. Appendix A, based on design and crash test experience, contains such guidance and provides insights that can be used in extrapolating the results of limited crash tests to the greater range of vehicles and impacts a railing will see in service.

C G 2.7.1.3

Note that the crash test matrix given in Table G2.7.1.3A brackets the lower end of the vehicle weight spectrum with an 1800-pound automobile and a 5400-pound pickup truck. Consideration was given to including a 3400-pound automobile, which represents a large segment of the vehicle population. Nevertheless, testing with this vehicle was omitted in the belief that testing with the other vehicles would be sufficient to avoid problems with the 3400-pound automobile. If there is reason to suspect that this is not true for a proposed design, testing with this vehicle might be considered. Wheel, bumper, or hood snagging are the problems most likely to be missed by the small automobile-pickup truck test combination. On the other hand, there is such a range in vehicle configurations that a problem with some 3400-pound cars might not be revealed by a particular 3400-pound vehicle chosen for a test. Probably the best way to avoid uncertainty about a railing's performance with various types of automobiles is to examine the physical features of many vehicles and use only designs that are not likely to be sensitive to variations in vehicle configurations.

A similar caution is appropriate for the design of railings to be used with sidewalks. Here, changing the impact angle, curb height, or sidewalk width will likely change the point at which an impacting vehicle will contact the railing. It is believed that the requirements set forth for testing railings to be used with sidewalks will reveal any vaulting problems. However, they may not be sufficient to reveal all potential snagging, so here again, the designer would be well advised to use a railing design that is not likely to be sensitive to variations in vehicle configuration or vehicle trajectory. Because the most cost effective railing design is likely to be one that is near its ultimate strength at its nominal performance limit, the designer should give particular attention to details such as rail splices, expansion joints, post and parapet attachments to the deck, and structural and geometric transitions, keeping in mind that relatively large deflections are to be expected at a railing's performance limit. All rail and parapet joints should provide shear and flexural continuity and, with rare exceptions, tensile continuity. One of these exceptions might be where very large superstructure expansion must be accommodated. Here special details should be developed.

C G 2.7.1.2.2

The primary function of a railing separation between highway traffic and a walkway or bikeway is the protection of pedestrians or cyclists from highway traffic. An adequate traffic railing will meet this requirement. However, the back side of a traffic railing used for such a separation must be compatible with the passage of pedestrians and cyclists. The requirement for a minimum 24-inch railing height and smooth surface on the walkway or bikeway side of a separator is believed to be sufficient for nearly all pedestrian needs and to prevent pedal snagging for cyclists. However, it is conceivable that some cyclists might lose their balance and fall over such a low railing. Thus, a designer should consider the expected volume of bike traffic, the closeness of highway traffic to the separator, highway sight distance needs, etc., and decide if a higher railing is appropriate. Since the risk associated with falling over a separator will usually be much less than that of falling off a bridge, this article allows the designer to select a railing height lower than required for pedestrian or bicycle railings. Assuming a separator is not needed to prevent pedestrians or cyclists from trespassing on the highway, the only place a height greater than the minimum 24 inches is likely to contribute to safety is where the risk to cyclists is significant. Here, a railing height much below 42 inches would probably not offer much improvement in safety, but a height of 42 inches would place the top of the railing where it could be easily grasped by a falling cyclist. Thus, a 42-inch railing height on the bikeway side of a separator is likely to be sufficient under nearly all conditions.

C G 2.7.1.3

The performance levels and selection procedures given in these guide specifications are based on consideration of the probabilities a bridge railing will be subjected to given impact conditions, the consequence of those impacts, given that various performance level railings are in place, and the cost of providing the various performance level railings. The objective is to match bridge railing performance level, and therefore cost, to site needs. Information on the development of the performance level selection procedures is given in Appendix B.

C G 2.7.1.3.1

The term "construction-year average daily traffic" (ADT) used in the performance level selection procedures described in this article applies to the estimated ADT that will be on a bridge in the year it is opened to traffic. However, the assumption is made that if this exceeds 10,000 vehicles per day per lane, traffic congestion will be such that traffic speeds will be restricted for a small part of the day when the highway is opened to traffic and any further increase in traffic will increase the portion of the day speed is restricted because of congestion. It is further assumed that as the traffic increases the increase in off-peak, free flow traffic will be offset by traffic running at reduced speeds in the peak periods, thus, preventing an increase in design level impacts to a railing. This assumption is only considered valid where the design impacts are based on design speeds of 50 mph or greater.

Note that the performance level selection procedure is very sensitive to design speed. (Design speed in these guide specifications is intended to have the same meaning as it does elsewhere in highway design). The design speeds in Table G2.7.1.3B assume the average speed under unrestricted, low volume flow conditions will be approximately 90 percent of the design speed.

There may be conditions where the free flow, off-peak speeds will differ significantly from those usually associated with a normal highway design speed. In some locations, the nominal design speed may be low, yet the roadway surface and alignment may have no limiting effect on speed. At locations where the highway design speed is not likely to be indicative of higher off-peak highway speeds, a railing design speed higher than the nominal highway design speed definitely should be used.

In other locations, the round-the-clock traffic volumes and the highway setting may keep off-peak speeds from reaching those usually associated with the nominal highway design speed. When this occurs, a railing design speed lower than the highway design speed may be appropriate. The choice of the railing design speed should be based on a careful study of site conditions.