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Chapter Forty-nine

ROADSIDE SAFETY

49-1.0 GENERAL

49-1.01 Clear-Zone Concept

The ideal roadway should be free from obstructions or hazardous conditions within the entire right of way. This is not practical due to economic, environmental, or drainage needs. The clear-zone concept was developed as a guide to determine how much obstruction-free recovery area should be provided for a run-off-the-road vehicle. The clear-zone width estimates provided herein, as derived from the AASHTO *Roadside Design Guide*, provide adequate space for approximately 80% of the motorists who run off the road to gain control of their vehicles. The clear-zone widths are only approximate values. It is the designer's responsibility to use engineering judgment, based on accident data when available, to determine if hazardous roadside features, including those outside the clear zone, warrant some type of treatment.

49-1.02 Situation Requiring Greater Clear-Zone Width

The basic clear-zone value assumes a tangent roadway section and level or near-level roadside slopes. A steeper downslope requires a greater clear-zone width because a vehicle requires more distance to stop or turn on a downslope. Therefore, the horizontal width of the clear zone on a downslope must be extended to be equivalent to a level clear zone. Likewise, a sharp horizontal curve, the location of a non-traversable drainage ditch, or a similar situation affects the area alongside the roadway defined as a recovery area for an errant vehicle. It is equally apparent that a slower-speed vehicle encroaching upon the roadside would not travel as far from the edge of the travel lane as one operating at a higher speed.

49-1.03 Applicability

The clear-zone requirements provided herein apply only to a project on a new location, a reconstruction project, or a 3R or partial 4R project on a freeway. The roadside-safety requirements for a 3R non-freeway project are provided in Section 55-5.0.

Where reference is made to speed, it is intended that the design speed be used. Design speed for a new construction or reconstruction project is provided in Chapter Fifty-three. Design speed for a 3R or partial 4R project on a freeway is provided in Chapter Fifty-four.

Where reference is made to AADT, it is intended to be the design-year traffic volume, which is assumed to be 20 years into the future. See Section 40-2.02.

49-1.04 Right of Way

Right of way is established to clear the construction limits. The construction limits are determined using a cross section that is traversable out to the right-of-way line or to the end of the clear zone, whichever is closer to the edge of the travel lane. Reducing right-of-way width by designing a steep embankment slope that will require the installation of guardrail should be avoided unless necessary due to restricted conditions (e.g., environmental, dense development).

49-1.05 Cost-Effectiveness of Safety Improvements

Warrants for countermeasures should be in accordance with the appropriate sections in this Chapter. The cost-effectiveness of the countermeasures for hazardous roadside conditions should desirably be considered. Therefore, the designer is encouraged to use the ROADSIDE computer program described in Section 49-10.0 as a tool in selecting an alternative safety treatment which offers the greatest anticipated return of safety benefits for the funds expended. ROADSIDE can be used to evaluate many of the safety treatments outlined in this Chapter to determine if they are cost effective at a specific location. ROADSIDE should not be used to determine whether or not countermeasures are warranted at a particular location. Engineering judgment must be used in applying the results from ROADSIDE.

49-1.06 Adherence to Design Criteria

An effort should be made to satisfy the design criteria provided in this Chapter (e.g., clear zone, barrier length of need). However, if this is not practical, a Level Two design exception is required. If the design criteria have not been satisfied, a brief rationale for not satisfying the criteria should be documented in the project file. It will not be necessary to prepare in-depth documentation to justify the decision. ROADSIDE can be used as part of the required documentation justification. Section 40-8.0 further describes the design-exception procedures.

Each new installation of a barrier device, barrier end treatment, transition device, or other safety hardware should satisfy the placement and installation criteria described in this Chapter and the INDOT *Standard Drawings*.

Environmental mitigation measures should not supersede roadside safety criteria. However, environmental mitigation features may be considered and incorporated into the project consistent with the criteria provided in this Chapter.

49-2.0 ROADSIDE CLEAR ZONE

49-2.01 Clear-Zone Width

Figure 49-2A, Clear-Zone Width for New Construction or Reconstruction, provides the clear-zone width for design. This is an estimate of the traversable area required adjacent to the edge of the travel lane and is based on a set of curves from the AASHTO *Roadside Design Guide*. These curves are for a tangent section and various side slopes. They were developed assuming an infinite length of side slope and 12-ft shoulders. Intervening ditches or multiple slopes require further consideration.

By referring to Figure 49-2A for a given side slope and design year AADT, the appropriate clear-zone width for a given design speed can be determined. For example, for a highway with a design speed of 60 mph, an AADT of 5000 vehicles and a 4:1 fill slope, the suggested clear-zone width is 40 ft. For a 4:1 cut slope, the required clear-zone width is 20 ft.

A basic understanding of the clear-zone concept is critical to its proper application. The value obtained from Figure 49-2A implies a degree of accuracy that does not exist. The values are based on limited empirical data which was then extrapolated to provide data for a wide range of conditions. Thus, the values obtained are neither absolute nor precise. They do, however, provide a frame of reference for making decisions on providing a safe roadside area.

In applying the clear-zone-width value, the designer should consider the following.

1. Context. The clear-zone width shown in Figure 49-2A is not absolute. It is desirable to eliminate all hazards within the right of way. However, this may not be practical because of economic or environmental constraints. It can be reasonable to leave a fixed object within the clear zone. An object beyond the clear-zone width may otherwise warrant removal or shielding. The use of an appropriate clear-zone width is a compromise between maximum safety and minimum construction costs. The designer should use engineering judgment in determining if a roadside hazard should be removed or shielded if it is outside the clear zone but within the right of way.

2. Adjustments. The clear-zone-width value shown in Figure 49-2A can be used for a roadway with shoulders of less than 12.0 ft in width without applying adjustment factors. The clear zone is measured from the edge of the travel lane, and slope averaging starts at the edge of shoulder.

3. Right of Way. If the clear-zone width extends beyond the right-of-way width, use the distance from the edge of the travel lane to the right-of-way line as the clear-zone width.

4. Guardrail. Where guardrail is required, the clear-zone width is used to determine the length of guardrail need.

5. Boundary. The clear-zone width should not be used as a boundary for introducing a roadside hazard such as a bridge pier, non-breakaway sign support, utility pole, or landscape feature. These should be placed as far from the roadway as practical.

6. Design Year AADT. The Design Year AADT will be the total AADT on a two-way roadway or the directional AADT on a one-way roadway. Examples of a one-way roadway include a ramp, or each directional roadway of a divided highway.

49-2.02 Clear-Zone-Width Adjustments

The clear-zone width should not vary with small variations in highway features. It should be constant on a length of road with a fairly consistent roadside. For a highway on new location, the clear-zone width should be constant for as much of the length of project as practical.

49-2.02(01) Horizontal-Curve Correction

A horizontal curve increases the angle of exit from the roadway and thus increases the width of clear zone required. Figure 49-2B, Clear-Zone-Width Adjustment Factor, K_{cz}, for Horizontal Curve, provides horizontal-curve correction factors that should be applied to the tangent clear-zone width to adjust it for roadway curvature. Figure 49-2C, Clear-Zone Transition for Curve Adjustment, Radius ≤ 3000 ft, illustrates the application of the adjusted clear-zone width on a curve. A curve with a radius of greater than 3000 ft as measured along the roadway centerline will not require a curvature adjustment. The horizontal-curve correction is required for a new construction or reconstruction project, or a 3R or partial 4R freeway project. If the correction cannot be practically applied, a Level Two design exception will be required.

The transition between different-width clear zones along a tangent and a curve with radius greater than 3000 ft should be applied as shown in Figure 49-2D, Clear-Zone Transition for Tangent Section or Curve with Radius > 3000 ft. The transition lengths between the beginning and the end of the narrower and wider clear zones may vary.

* * * * * * * * * *

Example 49-2.1

Given:	Rural Collector Design Speed = 55 mph Design-Year AADT = 2,000 Horizontal curve with a radius of 2000 ft 3:1 cut slope
Problem:	Find the adjusted clear-zone width.
Solution:	From Figure 49-2A, the clear-zone width on the tangent, CZ_t = 15 ft From Figure 49-2B, the curve correction factor, K_{cz} = 1.2 Clear-zone width for the curve, CZ_c = 15 ft *x* 1.2 = 18 ft The transition length, L =3.1 *x* 55 = 171 ft

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49-2.02(02) Slope Averaging

Variable-fill slopes can be used along a roadway to provide a relatively flat recovery area immediately adjacent to the roadway followed by a steeper side slope. Clear-zone widths for an embankment with variable side slopes ranging from essentially flat to 4:1 may be averaged, using a weighted average within the clear zone, to produce a composite clear-zone width. The slope averaging should begin at the outside edge of the adjacent travel lane for opposing traffic. See Figure 49-2E, Slope-Averaging Example.

For a slope flatter than or equal to 10:1, a slope of 10:1 is used for slope averaging.

Slope averaging applies only to slopes in the same direction. Slopes which change from a downslope to an upslope, as for a ditch section, cannot be averaged and should be treated as discussed in Section 49-2.03(01).

49-2.03 Clear-Zone Applications

49-2.03(01) Roadway with Shoulders or Sloping Curbs and V ≥ 40 mph

This Section applies to each project on a freeway, including 3R or partial 4R work, or to each new construction or 4R project on a rural or urban arterial, or a rural or urban collector with a design speed of 40 mph or higher. Section 49-2.03(02) provides the clear-zone application for a rural or urban collector with a design speed of 35 mph or lower, a rural local road, or an urban local street. Section 49-2.03(03) provides the clear-zone application for an urban facility with vertical curbs.

The designer should consider the following clear-zone applications.

1. Criteria. The clear-zone width provided in Figure 49-2A with the appropriate adjustments from Section 49-2.02 should be used.

2. Fill Slope for Reconstruction Project. To calculate the recommended clear-zone width, the following should be considered.

    a. Figures 49-2A and 49-2B, with the applicable design speed, AADT, and foreslope, are used to determine the appropriate clear-zone width. If the clear zone extends outside the right of way, use the right-of-way line as outside edge of the clear zone.

    b. For a variable fill slope of 4:1 or flatter, use a weighted average as discussed in Section 49-2.02(02) to determine the slope, then proceed as discussed in Item 2.a. above.

    c. A fill slope steeper than 4:1 is considered non-recoverable and should not be included in slope averaging. If a vehicle encroaches onto a non-recoverable slope, it can be assumed that the vehicle will continue to travel to the bottom of the slope. Therefore, if the clear-zone width extends onto the non-recoverable slope, a clear runout area should be provided at the bottom of the slope. This clear runout area should be equal in width to the portion of the clear-zone width which extends onto the non-recoverable slope or 12 ft, whichever is greater. See Figure 49-2F, Clear-Zone Application for Non-Recoverable Fill Slope, for an illustration of this procedure.

3. Fill Slope for New Facility. A 6:1 fill slope as shown in Figure 49-2G, Clear-Zone Application for Side Slope on New Facility, should be used adjacent to the roadway. At a minimum, the criteria described for a reconstruction project in Item 2 above may be used.

4. Cut Slope for Reconstruction Project. To calculate the recommended clear-zone width, the following should be considered.

    a. If a ditch is traversable, use Figure 49-2A with the applicable design speed and ADT to check the clear-zone width required for the foreslope and the backslope. The foreslope clear-zone width will control. However, if the toe of the backslope is within 10 ft of the shoulder edge, the clear-zone width for the backslope should be used. See Section 49-3.03 to determine if the ditch is traversable.

b. If the ditch is not traversable, the ditch should be reconstructed to a section which is traversable. The clear-zone width is then calculated as in Item 4.a above.

c. A cut slope of 2:1 is not desirable. However, if it will be retained or be constructed within the clear zone, the ditch in front of it should be made traversable. Figure 49-2H, Clear-Zone Application for Cut Slope (2:1 Backslope), illustrates the desirable cross section if a 2:1 backslope will be retained. If it is not practical to construct a traversable ditch, the designer should consider the accident experience at the site and use engineering judgment to determine if guardrail is warranted.

5. <u>Cut Slope for New Facility</u>. A ditch section as shown in Figure 49-2G should be used. At a minimum, the criteria described in Item 4 above for a reconstruction project may be used. However, 2:1 backslopes should not be used on a new facility.

6. <u>Auxiliary Lane</u>. Existing slopes adjacent to an acceleration or deceleration lane should be measured by averaging the slopes from the edge of the theoretical 12 ft shoulder. The clear-zone width is measured from the edge of the through travel lane, and is based on the mainline AADT and design speed. The clear-zone width should also be checked for the auxiliary lane using the auxiliary-lane AADT and mainline design speed. For the latter situation, the clear-zone width is measured from the outside edge of the auxiliary lane. Example 49-2.2 illustrates an example calculation of the clear-zone width from the edge of the through lane using slope averaging. Figure 49-2 I, Clear-Zone Application for Auxiliary Lane or Ramp, illustrates the clear-zone application for an auxiliary lane next to the mainline.

7. <u>Ramp</u>. If the obstacle is adjacent to a ramp, the clear-zone width should be determined the same as for the mainline, using the AADT and design speed of the ramp and the slope from the ramp shoulder. Figure 49-2 I illustrates the clear-zone application for a ramp/mainline configuration.

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<u>Example 49-2.2</u>

Given: Rural freeway with an exit ramp
Design-Year AADT = 7,000
Design speed = 70 mph
A 12-ft wide deceleration lane with an 8-ft right shoulder
A 4:1 slope adjacent to deceleration lane shoulder

Problem: Determine the clear-zone width adjacent to the deceleration lane.

Solution: Start slope averaging from edge of theoretical shoulder; see Figure 49-2J, Clear Zone / Slope Average, Example 49-2.2.

First Trial: Assume that clear-zone width for the mainline ends 10 ft beyond the deceleration lane shoulder.

Therefore, assumed clear-zone width = 12 + 8 + 10 = 30 ft

$$\text{Slope} = \frac{(8)(\text{-}0.04) + (10)(\text{-}0.25)}{18} = \frac{(\text{-}0.32) + (\text{-}2.5)}{18} = 0.16 \text{ or } 6:1 \text{ slope}$$

From Figure 49-2A, the clear-zone width = 35 ft

35 ft > 30 ft; therefore, a second trial is necessary with a wider assumed clear zone.

Second Trial: Assume that clear-zone width ends 20 ft from existing shoulder.

Therefore, assumed clear-zone width = 12 + 8 + 20 = 40 ft

$$\text{Slope} = \frac{(8)(\text{-}0.04) + (20)(\text{-}0.25)}{28} = \frac{(\text{-}0.32) + (\text{-}5)}{28} = 0.19$$

or approximately 5:1.

From Figure 49-2A, the clear-zone width = 38 ft

40 ft is close enough to 38 ft; therefore, 38 ft is the required clear-zone width from the edge of the through travel lane.

* * * * * * * *

Example 49-2.3

Given: Rural facility with flat-bottom side ditch
Design speed = 60 mph
Design-Year AADT = 1490

Problem: Determine adjusted clear-zone width after slope averaging, and if obstacle must be removed if within such clear zone. See Figure 49-2K, Clear-Zone / Slope Average, Example 49-2.3.

Solution:

1. To determine the clear-zone width for the foreslope in the side ditch, an average foreslope must be calculated. See Figure 49-2E for an example of foreslope averaging.

A ditch not having the desirable cross section (see Figure 49-3D, 49-3E, or 49-3F) should be located at or beyond the clear-zone limit. However, a backslope steeper than 3:1 is typically located closer to the roadway. If this slope is relatively smooth and unobstructed, it presents minimal safety problems to an errant motorist. If the backslope consists of a rough rock cut or outcropping, shielding may be warranted as discussed in Section 49-5.04.

The foreslope and the ditch-bottom slope should be averaged to obtain a weighted average foreslope run, $F_{w\,run}$, as follows:

$$F_{w\,run} = \frac{W_f + W_d}{W_f\left(F_{rise}/F_{run}\right) + W_d\left(D_{rise}/D_{run}\right)} \qquad \text{(Equation 49-2.1)}$$

Where: W_f = Width of foreslope, 10 ft
W_d = Width of ditch, 4 ft
F_{rise} = Foreslope rise, 1
F_{run} = Foreslope run, 6
D_{rise} = Ditch slope rise, 1
D_{run} = Ditch slope run, since flat, use 10

$$F_{w\,run} = \frac{10+4}{10(1/6)+4(1/10)} = \frac{14}{2.07} = 6.8$$

The 6.8 weighted foreslope run affects a 6.8:1 foreslope, which is flatter than 6:1.

2. Determine clear-zone width for flatter-than-6:1 foreslope (fill) from Figure 49-2A as 22 ft.

3. Calculate the percentage of the clear-zone width available from the edge of travel lane to the back of the ditch bottom, $CZ_{\%FD}$, as follows:

$$CZ_{\%FD} = \frac{100\left(W_s + W_f + W_d\right)}{CZ_F} \qquad \text{(Equation 49-2.2)}$$

Where: W_s = Width of shoulder, 6 ft
CZ_F = Clear-zone width for foreslope, 22 ft

$$CZ_{\%FD} = \frac{100(6+10+4)}{22} = 92\%$$

4. For a ditch within the desirable cross-section area shown in Figure 49-3D, 49-3E, or 49-3F, the clear-zone width may be determined from Figure 49-2A. However, where the clear-zone width exceeds the available clear-zone width for the foreslope, an adjusted clear-zone width may be determined as shown below.

   Determine clear-zone width for 4:1 backslope (cut) from Figure 49-2A as 16 ft.

5. Subtract $CZ_{\%FD}$ from 100%, divide by 100, and multiply the result by the clear-zone width for the backslope to obtain the required clear-zone width for the backslope, CZ_{BR}, as follows:

$$CZ_{BR} = \frac{CZ_B(100 - CZ_{\%FD})}{100} \qquad \text{(Equation 49-2.3)}$$

Where CZ_B = clear-zone width for backslope, 16 ft

$$CZ_{BR} = \frac{5(100 - 92)}{100} = 1.28 \text{ ft}$$

6. Add the available clear-zone width on the foreslope to CZ_{BR} to obtain the adjusted clear-zone width, CZ_{ADJ}, as follows:

$$CZ_{ADJ} = \frac{(CZ_{\%FD})(CZ_F)}{100} + CZ_{BR} \qquad \text{(Equation 49-2.4)}$$

$$CZ_{ADJ} = \frac{(92)(22)}{100} + 1.28 = 21.5 \text{ ft}$$

The obstacle is actually located 6 + 10 + 4 + 16, or 36 ft from the edge of travel lane. Since the adjusted clear-zone width is only 22 ft, the obstacle need not be removed. However, removal should be considered if this one obstacle is the only fixed object this close to the edge of travel lane for a significant length.

* * * * * * * *

49-2.03(02) Roadway with Shoulders or Sloping Curbs and V ≤ 35 mph

This Section applies to each new construction or reconstruction project on a rural or urban collector with a design speed of 35 mph or lower, or to a local road or street. The clear-zone width should be determined from Figure 49-2A, Clear-Zone Width for New Construction or Reconstruction, with the applicable adjustments. The minimum clear-zone width is 10.0 ft for a tangent section and should be adjusted as discussed in Section 49-2.02 for a horizontal curve. Where the clear-zone width extends onto a 3:1 fill slope, a clear recovery area as shown in Figure 49-2F, Clear-Zone Application for Non-Recoverable Fill Slope, should be provided.

49-2.03(03) Roadway with Vertical Curbs

For an urban arterial, collector, or local street with vertical curbs at either the edge of the travel lane or the edge of shoulder, the minimum clear-zone width is 10 ft from the edge of the travel lane or to the right-of-way line, whichever is less.

49-2.03(04) Appurtenance-Free Area

The roadway should have a 1.5 ft appurtenance-free area from the face of curb or from the edge of the travel lane if there is no curb. However, for a traffic-signal support, the appurtenance-free area should be 2.5 ft. The appurtenance-free area is defined as a space in which nothing, including breakaway safety appurtenances, should protrude above the paved or earth surface (see Figure 49-2L, Appurtenance-Free Zone). The objective is to provide a clear area adjacent to the roadway in which nothing will interfere with extended side-mirrors on trucks, with the opening of vehicular doors, etc.

49-2.03(05) On-Street Parking

The following clear-zone requirements will apply.

1. <u>Continuous 24-Hours Parking</u>. No clear zone is required where there is continuous 24-h parking, except that the appurtenance-free area of 1.5 ft should be provided from the face of the curb, or the edge of the parking lane if there is no curb.

2. <u>Parking Lane Used as a Travel Lane</u>. The clear-zone width should be determined assuming the edge of the parking lane as the right edge of the rightmost travel lane.

49-3.0 TREATMENT OF OBSTRUCTIONS

<u>49-3.01 Roadside Hazards</u>

49-3.01(01) Range of Treatments

If an obstruction or non-traversable hazard is determined to be within the clear zone, it should be treated, in order of preference, as follows:

1. removed or redesigned so that it can be safely traversed;
2. relocated outside of the clear zone to a point where it is less likely to be hit;
3. made breakaway to reduce impact severity;
4. shielded with a traffic barrier or impact attenuator; or
5. delineated if the above treatments are not practical.

49-3.01(02) Example Hazards

The method for treating an obstruction should be based on an analysis of factors such as initial cost, maintenance cost, and the greatest safety return. The following is a list of some of the obstructions and hazards which should be considered for treatment.

1. non-breakaway sign support or luminaire support. A sign or luminaire in the clear zone should not be placed on a breakaway support if there is a sidewalk and there is a potential for the support falling on a pedestrian or bicyclist;

2. bridge pier;

3. bridge-railing end. A bridge-railing end must have appropriate approach shielding whether or not the end is outside the clear zone;

4. the end of each culvert which is transverse to the mainline road and does not have acceptable end treatments in accordance with Section 49-8.01;

5. concrete headwall for a culvert;

6. tree;

7. retaining-wall end;

8. mailbox support. A mailbox support should be placed in accordance with the INDOT *Standard Drawings*, INDOT *Standard Specifications*, and Section 51-11.0;

9. wood pole or post with a cross sectional area greater than 0.15 ft^2;

10. utility pole. A utility pole should be installed as close as practical to the right-of-way line;

11. steel pipe with an inside diameter greater than 2 in;

12. large boulder;

13. rough rock cut;

14. bridge-cone slope that is 2:1 or steeper and can be hit head-on;

15. severely rutted or eroded slope;

16. transverse embankment slope for a drive, public road approach, ditch check, or median crossover that is steeper than shown in Figure 49-3A, Transverse Slopes, for the selected design speed and AADT level;

17. ditch cross-section that is not in accordance with the criteria described in Section 49-3.02;

18. stream or body of water where the permanent water depth is 2 ft or greater; or

19. slope steeper than 1:1 at the edge of shoulder and a height greater than 2 ft.

49-3.02 Embankment

The factors in determining the need for a roadside barrier at an embankment are the lateral clearances from the barrier to the hazard and from the barrier to the top of the embankment slope. They are based on distances from the face of the barrier, considering the rail-blockout-post thickness and the barrier deflection properties.

The Figures 49-3B series describes the barrier warrant at an embankment for a 2-lane 2-way roadway for a design speed of 35, 40, 45, 50, 55, 60, or 70 mph, respectively. Figure 49-3C describes the barrier warrant at an embankment for a divided or undivided roadway of 4 or more lanes. Though these figures were developed using 12-ft lanes and 10- to 12-ft shoulders, they can be used for another lane or shoulder width. A barrier at an embankment is not warranted on a facility with a design speed of 30 mph or lower. Slope-height combinations which appear on or below the curve do not warrant shielding. To adjust for horizontal curvature and grade, use the factors shown in Figure 49-6B, Grade Traffic-Adjustment Factor, K_g, and Curvature Traffic-Adjustment Factor, K_c. The following example illustrates how to use the embankment-barrier warrant figures.

* * * * * * * * *

Example 49-3.1

Given: 2-lane, 2-way highway
Design Speed = 55 mph
Design Year AADT = 3000
Tangent Section
Grade = 2%
Foreslope = 2.0:1
Fill Height = 10 ft

Problem: Determine if guardrail is warranted at the embankment.

Solution: Using Figure 49-3B(55), it can be determined that a barrier is not warranted based on the embankment height. However, the need for a barrier should be considered based on other factors (e.g., nearby hazards, accident history).

* * * * * * * *

Example 49-3.2

Given: Same highway section as discussed in Example 49-3.1, but with a horizontal radius of 820 ft, the embankment of concern on the outside of the curve, and a fill height of 10 ft.

Problem: Determine if a barrier is warranted at the embankment.

Solution:

1. The Design Year AADT first must be adjusted by the horizontal curvature factor K_c = 4.0 from Figure 49-6B, Grade Traffic-Adjustment Factor, K_g, and Curvature Traffic-Adjustment Factor, K_c.

   Corrected Design Year AADT = 3,000 x 4.0 = 12,000

2. Using Figure 49-3B(55), it can be determined that a barrier is now warranted based on the embankment height.

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49-3.03 Roadside Ditch

49-3.03(01) General Guidelines

Traversable-ditch cross sections are defined in Figure 49-3D, Preferred Ditch Cross Section, Width < 4 ft; Figure 49-3E, Preferred Ditch Cross Section, 4 ft ≤ Width ≤ 8 ft; and Figure 49-3F, Preferred Ditch Cross Section, Width > 8 ft. Two curves are shown on each figure. The area below the lower curve represents a ditch cross section which can be traversed by a vehicle containing unrestrained occupants and, thus, is considered to be desirable. A ditch cross section which is between the upper curve and the lower curve is considered to be acceptable. However, vehicle occupants must be restrained in order to safely traverse the ditch. Minor encroachment into the area above the upper curve may be necessary due to right-of-way restrictions or to avoid nominal changes the existing ditch. In addition, the following should be considered.

1. A slope of 3:1 should be used only where site conditions do not permit the use of a flatter slope.

2. To permit traversability of a 3:1 slope, embankment surfaces should be uniform. Vehicular rollover can be expected if the embankment is soft or rutted.

3. A foreslope steeper than 4:1 is not desirable because its use severely limits the range of backslopes producing a safe ditch configuration.

49-3.02(02) Application

If a ditch is outside the clear zone, it need not be checked for traversability. For a ditch within the clear zone, the following describes the appropriate application of Figure 49-3D, 49-3E, or 49-3F.

1. <u>In Fill, Reconstruction Project</u>. Existing ditch-slope combinations which are within the desirable or acceptable range may be retained. An area with ditch slope combinations which are not within the undesirable range should be evaluated for cost and accident history before deciding to make an improvement. If an improvement is warranted, the slope combination should preferably be within the desirable range and at least within the acceptable range.

2. <u>In Fill, New Facility</u>. A foreslope, backslope, and ditch width should be selected that will be within the desirable range shown in Figure 49-3D, 49-3E, or 49-3F.

3. <u>In Cut, Reconstruction Project</u>. If the ditch is such that to flatten the slopes or move the ditch out farther means acquiring more right-of-way, this should be done only if considered to be cost effective. Other means of making the ditch traversable can be evaluated as follows:

    a. use of a pipe in the ditch;

b. raise the grade of the ditch; or

c. place uniform riprap in the ditch.

4. In Cut, New Facility. The desirable ditch section is shown in Figure 49-2G, Clear-Zone Application for Side Slope. For a minimum ditch section, a section should be provided which is within the desirable range shown in Figure 49-3D, 49-3E, or 49-3F.

49-3.04 Drainage Structure [Rev. Sept. 2012]

49-3.04(01) Drainage Structure Perpendicular or Skewed to Roadway Centerline

The following provides the criteria for a drainage structure which is perpendicular or skewed to the roadway centerline. The point at which the top of the structure protrudes from the slope is within the clear zone.

1. 12-in. Diameter Culvert. This type of structure or equivalent pipe-arch culvert should include a metal culvert end section as shown on the INDOT *Standard Drawings*.

2. 15-in. to 60-in. Diameter Culvert, 10-deg Skew or Less. This type of structure or equivalent pipe-arch culvert should be installed with a safety metal culvert end section, or an optional grated box end section (GBES), as shown on the INDOT *Standard Drawings*. For a site with side slopes of 3:1 or steeper, a culvert of 15 in. to 30 in. diameter may include a safety metal culvert end section. For a site with a side slope of 3:1 or steeper, a culvert of 36 in. to 60 in. diameter may include a safety culvert metal end section or a GBES. A GBES type I should be used at a high-accident location where it is anticipated that a vehicle will most likely traverse it based on previous accident experience. This does not apply to where the culvert end is shielded with adequate length to shield the end from an errant vehicle.

3. 15-in. to 60-in. Diameter Culvert, Greater Than 10-deg Skew. This type of structure or equivalent pipe-arch culvert should have a GBES installed perpendicular to the roadway centerline as shown on the INDOT *Standard Drawings*. This applies except where the culvert end is shielded with adequate length to shield the end from an errant vehicle. A large skew may require the use of a GBES that is intended for a larger pipe in order to provide an adequate opening in the GBES for the skewed pipe.

   It may be necessary to maintain the direction of flow in a straight line at the inlet and the outlet in order to perpetuate the channel flow. The GBES must be installed parallel to the pipe centerline, and the roadway embankment must be warped around the GBES to present a smooth slope profile.

4. <u>66-in. or Larger-Diameter Culvert</u>. If the point at which the top of this type of culvert, pipe structure, or equivalent pipe-arch protrudes from the slope is within the clear zone, shielding should be provided. See Figure 49-3G, Large-Culvert End within Clear Zone. If the culvert end is outside the clear zone, shielding should be placed to protect an errant motorist from the culvert end. If there is inadequate cover over the culvert to drive guardrail posts, it will be necessary to use the detail for shielding over a low-fill culvert as shown in Section 49-5.05 and the INDOT *Standard Drawings*.

5. <u>Pipe in the Median</u>. The adjoining ends of two transverse culverts in the median between divided travelways or between a main road and a frontage road should be connected if the ends are within the clear zone. At a minimum, a pipe in the median should be treated the same as described above. However, a pipe structure of 15 in. through 60 in. diameter should have a GBES type I. A culvert with appropriate sloped grates should be installed in the parallel ditch as shown in Figure 49-3H, Culvert End Treatment, Median Section.

6. <u>Box Culvert or Three-Sided Structure</u>. See Figure 49-3 I, Clear Zone / Barrier at Culvert, for acceptable options. The most cost-effective treatment should be considered.

   Removing sections of a box culvert and attaching metal circular or pipe arch adapters, a short section of metal culvert, and then a GBES is also an option if the span is less than or equal to 5 ft.

49-3.04(02) Drainage Structure Parallel to Roadway Centerline

The following provides the criteria for a drainage structure which is parallel to the roadway centerline and is within the clear zone.

1. <u>12-in. to 60-in. Diameter Culvert in the Median</u>. This type of pipe structure under a median crossover should be end-fitted with GBES type II with a slope satisfying the criteria shown in Figure 49-3A, Transverse Slope.

2. <u>12-in. Diameter Culvert</u>. This type of pipe structure or equivalent pipe-arch culvert should include the metal culvert end section as shown on the INDOT *Standard Drawings*.

3. <u>15-in. to 60-in. Diameter Culvert in Side Ditch</u>. This includes both ends of a culvert adjacent to a two-way roadway where both ends are within the clear zone for both the adjacent and opposing traffic. This also includes the end-facing oncoming traffic on the outside of a divided highway. It does not apply to the traffic downstream end of a culvert if it is outside the clear zone for opposing traffic. See Figure 49-3J, Culvert End Treatment, Longitudinal Structure.

This type of pipe structure should be installed with a safety metal culvert end section. If a 10:1 slope is required parallel to the roadway, the 10:1 slope may be warped to match the 6:1 slope of the safety metal culvert end section. GBES type II, with a slope as shown in Figure 49-3A, should be used at each high-accident location where it is anticipated that a vehicle will most likely traverse it based on previous accident experience. This does not apply where the culvert end is shielded with adequate length to shield the end from an errant vehicle.

49-3.04(03) Drainage Inlet

The following provides the criteria for the placement of a drainage inlet within the clear zone.

1. General. A type 7 inlet with vertical projections of 4 in. or greater should not be used in a new installation. An existing type 7 inlet should not be replaced unless its location is considered to be a safety hazard.

2. Reconstruction Project. A type E-7 inlet in a median should not be replaced unless its location is considered to be a safety hazard. The type E-7 inlet should be replaced with an acceptable inlet type if the slopes adjacent to it must be re-graded to eliminate a hazardous depression. If an existing type E-7 casting is broken, it should be replaced.

3. New Facility or Reconstruction Project. Only a type N-12 or P-12A inlet will be permitted, as follows:

    a. in a median in advance of the 20:1 slope grading for an attenuation device at a median pier or overhead sign structure support; or

    b. in a side ditch in advance of the 20:1 slope grading for a guardrail run that is buried in a backslope.

4. Interstate Route. A type N-12 or P-12A inlet that does not have a 10:1 slope and is parallel to the centerline should be replaced with a new 10:1 slope type N-12 or P-12A inlet as shown on the INDOT *Standard Drawings*.

49-3.05 Curbs

49-3.05(01) General

The use of curbs should be avoided. However, they can be necessary to control drainage or to protect erodible soils. Section 45-1.05 and the INDOT *Standard Drawings* provide detailed information on the warrants and types of curbs used. If considering curbing relative to roadside safety, the following should be considered.

1. Design Speed. A facility with a design speed of 50 mph or higher should be designed without curbs. However, if necessary, a 4-in. sloping curb may be used. A facility with a design speed of 45 mph or lower may use either a sloping or vertical curb.

2. Roadside Barrier. The use of a curb with a roadside barrier is discouraged and, specifically, a curb higher than 4 in. should not be used with a barrier. Terrain conditions between the traveled way and a barrier can have significant effects on barrier performance. Curbs and a sloped median (including superelevated section) are two prominent features which deserve attention.

3. Redirection. Curbs offer no safety benefits to vehicular behavior following impact on a high-speed roadway. Therefore, a curb should not be used for the purpose of redirecting an errant vehicle.

49-3.05(02) Curbs on a Ramp

Existing curbs on a ramp should be removed and new stabilized shoulders should be constructed. Using 16 ft as the pavement width for the ramp, the shoulders should be constructed such that a 4-ft desirable, 2.5-ft minimum width stabilized shoulder is on the left side and an 8-ft desirable, 7.5-ft minimum width stabilized shoulder is on the right side. If the existing pavement is more than 16 ft in width, that portion of the existing pavement over 16 ft should be considered as part of the shoulders. For a new facility, see Section 48-5.0 and the INDOT *Standard Drawings*.

49-3.06 Bridge Pier and Spillslope

49-3.06(01) New-Construction Project

The following provides the criteria for bridge-pier or spillslope clearance for a new-construction project:

1. Divided Highway. The spillslope clearance should be equal to the clear-zone width of the approach roadway.

2. Vertical Clearance. After establishing the clear-zone width beneath an overhead structure, the critical vertical clearance must be determined. A critical vertical clearance of 14 ft should be provided at the edge of the clear zone. The slope between the edge of shoulder and the edge of clear zone should not be steeper than 6:1. If the slope is steeper than 6:1, it

should be flattened to 6:1 to provide a greater vertical clearance. See the following examples.

a. Example 1. A county road crosses over a tangent freeway having a design speed of 70 mph and a design-year projected AADT of 7500. From Figure 49-2A, Clear-Zone Width for New Construction or Reconstruction, the minimum clear-zone width to the face of pier or toe of the 2:1 spillslope, assuming a 6:1 approach fill slope, is 35 ft. See Figure 49-3K, Bridge Pier or Spillslope Clearance, New Construction, illustration (A). To maintain a minimum 14-ft vertical clearance at the outer edge of the clear zone, the maximum permissible upward slope beyond the shoulder is 8:1 (cut section).

b. Example 2. A county road crosses over a superelevated roadway having a design speed of 60 mph, a design-year projected AADT of 1200, and a horizontal curve with a 1500-ft radius. To hold the 14-ft minimum vertical clearance at the outer edge of the clear zone, the maximum permissible slope beyond the shoulder line is 6:1 (upward) and 10:1 (upward) on the high side. See Figure 49-3K, illustration (B).

   Basic clear-zone width of approach roadway:
      low side, 6:1 fill = 25 ft (Figure 49-2A)
   Basic clear-zone width of approach roadway:
      high side, 6:1 fill = 25 ft (Figure 49-2A)
   Horizontal-curve correction factor = 1.4 (Figure 49-2B)
   Horizontal clearance to pier or toe of 2:1 spillslope (low side) = 25 ft
   Horizontal clearance to pier or toe of 2:1 spillslope (high side)
      = 25 ft *x* 1.4 = 35 ft

   The curve correction factor is applied only to the outside (high side) of a horizontal curve.

2. <u>Shoulder-Pier Clearance</u>. The use of a shoulder pier should be avoided if possible. However, if it is considered necessary, it should be placed as far from the edge of the traveled way as practical and shielded as described in Section 49-5.04, if located within the clear zone.

3. <u>Median Pier</u>. A median pier should be shielded in accordance with the INDOT *Standard Drawings*.

49-3.06(02) Reconstruction Project

If a pier or a bridge-cone spillslope is within the clear zone, the following procedures apply.

1. Slopewall Set Back 30 ft from Edge of Travel Lane. Establish the elevation of the bottom of the slopewall. Below this elevation, the upstream bridge cone should be graded at a downward slope equal to the slope below the concrete slopewall to the intersection with the natural ground. This slope should be constructed between the edge of the asphalt paved apron and as close as practical to the right-of-way line. The built-up slope should be transitioned to the existing ground near the right-of-way line at a 4:1 or flatter slope. See Section 49-3.04 for drainage-structure end-treatment requirements.

   The area between the end of the slopewall, and bounded by the edge of the paved shoulder and the base of slopewall, should be paved. At the downstream end of the paved apron, the new embankment should be graded at a 6:1 downward slope to approach the existing ground. Typical details are provided in Figure 49-3L, Treatment at Existing Bridge Cone, Slopewall ≥ 30 ft from Travel Lane.

2. Slopewall Set Back Less Than 30 ft from Edge of Travel Lane. A spillslope located less than 30 ft from the travel lane should be graded in accordance with Figure 49-3M, Treatment at Existing Bridge Cone, 10 ft ≤ Slopewall < 30 ft from Travel Lane. The upstream bridge cone should be graded at a downward slope to intersect the natural ground. This slope should be constructed between the edge of slopewall and as close as practical to the right-of-way line; see Figure 49-3M. The built-up slope should be transitioned to the existing ground at a 4:1 or flatter slope. See Section 49-3.04 for culvert end-treatment requirements. At the downstream end, the embankment should be graded at a 6:1 downward slope to meet the existing ground.

49-3.06(03) Longitudinal Side-Slope Transition

If it is necessary to transition a side slope, the transition should be made such that the maximum longitudinal slope (with regard to the grade line) along the roadside does not does not taper at less than 30:1. The 30:1 taper should be based on the sideslope elevation differences at the edge of each respective clear zone.

For example, a transition may be needed from a 6:1 fill slope to a 6:1 cut slope at a bridge overpass. This should be accomplished over a distance calculated as follows:

1. Given: Design Speed = 70 mph, Design-Year AADT = 7500.

2. Distance to shoulder slope break = 11 ft from edge of traveled way

3. Elevation differential from slope break for 6:1 fill slope at 35 ft is as follows:

$$\left(\frac{35 - 11}{6}\right) = 4\ ft^3$$

4. Elevation differential from slope break for 6:1 cut slope at 35 ft is as follows:

$$\left(\frac{35 - 11}{6}\right) = 4\ ft^4$$

5. Change in elevation along roadside at clear zone limits = 4 ft + 4 ft = 8 ft.

6. Transition distance at 30:1 longitudinal slope = 8 *x* 30 = 240 ft.

Therefore, the transition from the 6:1 fill slope to the 6:1 cut slope should occur over approximately a 240-ft distance along the roadway.

49-3.07 Signing, Lighting, or Signalization

The following provides the roadside-safety criteria for a sign support, or lighting or signal pole within the clear zone.

1. Exit Sign in Gore Area. An exit gore sign should be placed in each gore area, though outside the paved portion of the gore, on an expressway or freeway as shown on Figure 49-3 O, Gore-Area Treatment.

2. Breakaway Supports. The stub of a breakaway sign or lighting support, or substantial remains of a barrier end-treatment post, which are intended to remain after the unit has been struck, should have a maximum projection of 4 in. See Figure 49-3P, Breakaway Support Stub Clearance Diagram, or Figure 49-3Q, Light-Standard Treatment, Fill Slope 4:1 or Steeper.

3. Ground-Mounted Sign. Supports for a ground-mounted sign should be breakaway or yielding, except those behind an adequate length of barrier to protect an errant motorist from the sign support, or those within a sidewalk. New sign supports behind a barrier should have adequate clearance from the back of the barrier post to provide for the barrier's dynamic deflection; see Section 49-4.01(03).

4. Lighting. A conventional light standard should be breakaway except that within a sidewalk. A breakaway light standard (except that shielded by a barrier) should not be placed where the opportunity exists for it to be struck more than 9 in. above the normal point of vehicular bumper impact. Normal bumper height is 1.5 ft. To avoid a light standard being struck at an improper height, it should be placed, and the area around it graded, as follows:

a. Fill Slope Flatter than 6:1. There are no restrictions on location, nor is special grading required. A light standard should be placed 20 ft from the edge of the travel lane or 10 ft from the edge of shoulder.

b. Fill Slope of 5:1 or 6:1. Follow the grading plans shown in the INDOT *Standard Drawings*. A light standard should be placed 20 ft from the edge of the travel lane or 10 ft from the edge of shoulder.

c. Fill Slope of 4:1 or Steeper. A light standard should be offset 3 ft from the edge of shoulder or 12 ft from the edge of the travel lane, whichever is greater. Grading should be provided as shown in Figure 49-3Q.

d. Cut Slope. Follow the grading plans shown in the INDOT *Standard Drawings*.

An existing breakaway light standard should be evaluated to determine if it is necessary to relocate it, re-grade around its base, or upgrade the breakaway mechanism to current AASHTO standards. The determination of the extent of work necessary for treatment of an existing breakaway light standard involves a review of a number of variables. Therefore, this determination must be made by the Highway Management Design Division's Office of Traffic Review. If Federal-aid funds will be used for construction and the project is on the National Highway System and is not exempt from FHWA oversight, the FHWA should also be consulted.

5. High-Mast Lighting. High-mast lighting should be placed to provide a desirable clear-zone width of 80 ft. The minimum clear-zone width will be the roadway clear-zone width through the area where the high-mast lighting is located.

6. Traffic Signal. A traffic-signal support for a new-construction or reconstruction project should be placed to provide the roadway clear zone through the area where the traffic-signal support is located. However, the following exceptions will apply:

a. Channelizing Island. Installation of a signal support in a channelizing island should be avoided. However, if a signal support must be located in a channelizing island, a minimum clearance of 30 ft should be provided from all travel lanes (including turn lanes) in a rural area. Such minimum clearance should be provided in an urban area where the posted speed limit is 50 mph or higher. In an urban area where the island is bordered by barrier curb and the posted speed limit is 45 mph or lower, a minimum clearance of 10 ft should be provided from all travel lanes including turn lanes.

b. Non-Curbed Facility, Posted Speed Limit ≥ 50 mph or AADT > 1500. Where conflicts exist such that the placement of a signal support outside of the clear zone is impractical (e.g., conflicts with buried or utility cables), the signal support should be located at least 10 ft beyond the outside edge of the shoulder.

c. Non-Curbed Facility, Posted Speed Limit ≤ 45 mph or AADT ≤ 1500. Where conflicts exist such that the placement of a signal support outside of the clear zone is impractical (e.g., conflicts with buried or utility cables), the signal support should be located at least 6.5 ft beyond the outside edge of the shoulder.

7. Large Sign. A large sign of over 50 ft^2 in area on slipbase breakaway supports should not be placed where the opportunity exists for it to be struck more than 9 in above the normal point of vehicular bumper impact. Normal bumper height is 1.5 ft. To avoid such a sign being struck at an improper height, it should be placed as follows:

   a. Fill Slope 5:1 or Flatter. The sign should be located a minimum of 30 ft from the edge of the travel lane to the nearest edge of the sign.

   b. Fill Slope of 4:1 or Steeper. The nearest sign edge should be located 6 ft from the edge of shoulder or 12 ft from the edge of the travel lane, whichever is greater.

8. Roadside Appurtenances. Roadside appurtenances such as a large breakaway sign or lighting support should not be located in or near the flow line of a ditch. If these supports are placed on a backslope, they should be offset at least 10 ft up the slope from the bottom of the ditch.

Roadway signing and lighting plans for a project are often prepared separately by different INDOT designers or consultants. Therefore it is possible that guardrail, guardrail end treatments, impact attenuators, light standards, or breakaway overhead sign supports within the clear zone may have been located too close to one another and are therefore clustered at one location. An errant vehicle may have multiple impacts due to this clustering of such devices. The multiple impacts may cause higher G forces than those recommended in National Cooperative Highway Research Program *Report 350* (NCHRP 350), thus creating a hazardous condition for the occupants of the impacting vehicle.

Where the devices are clustered, they should be separated and relocated as far from one another as conditions permit to avoid the possibility of multiple impacts to them while ensuring that each system performs properly. For example, guardrail and end treatments may be relocated by extending each guardrail run beyond its length of need and then attaching the end treatment to the guardrail.

The project manager should coordinate the review of all separately-developed sets of plans with the designer of the mother project and the reviewer before the final design stage.

49-3.08 Miscellaneous Grading

Considerations to be made regarding grading are as follows:

1. Gore Area. A gore area should be graded with a slope of not steeper than 10:1 parallel to the roadway.

2. Median Cross Slope. For a reconstruction project, the median cross slope should be 4:1 at steepest, but desirably 6:1 or flatter. For a median cross slope on a new facility, see the INDOT *Standard Drawings*.

3. Shoulder Wedge. On a reconstruction project, a wedge on the outside and inside shoulders should be constructed as shown on Figure 49-3R, Shoulder Wedges.

4. Rock Cut. As indicated in Section 49-3.01(02), a rough rock cut located within the clear zone may be considered a roadside hazard. The following will apply to its treatment.

    a. Hazard Identification. There is no precise method to determine whether or not a rock cut is sufficiently ragged to be considered a roadside hazard. This will be a judgment decision based on each evaluation.

    b. Debris. A roadside hazard may be identified based on known or potential occurrences of rock debris encroaching onto the roadway.

    c. Barrier Warrant. If the rock cut or rock debris is within the clear zone, a barrier may be warranted.

49-4.0 ROADSIDE-BARRIER LATERAL OFFSET AND LONGITUDINAL EXTENT

A roadside barrier should be placed to protect an errant vehicle from an obstacle which is within the clear zone and cannot be removed, or where described in Section 49-3.0.

49-4.01 Lateral Placement

49-4.01(01) Barrier Offset

Some of the factors to consider in the lateral placement of a roadside barrier include the following:

1. clearance between barrier and hazard being shielded to allow for deflection of the barrier;

2. effects of terrain between the edge of the traveled way and the barrier on an errant vehicle's trajectory;

3. probability of impact with barrier as a function of its distance off the traveled way;

4. flare rate and length of need of transitions and approach barriers; and

5. the need to offset a barrier from the edge of shoulder so that the full shoulder width can be used. For new construction, the desirable offset is 2 ft from the effective usable-shoulder width. The minimum offset is 1 ft from the effective usable-shoulder width. For a reconstruction project, the desirable offset is 2 ft from the effective usable-shoulder width. The minimum offset is 0 ft from the effective usable-shoulder width. However, if the design-year AADT exceeds 100,000, the offset should be 2 ft from the effective usable-shoulder width.

A roadside barrier should be placed as far from the traveled way as conditions permit, thereby minimizing the probability of impact with the barrier. The barrier should be placed beyond the shy line offset; see Section 49-4.02(01).

The practicality of offsetting the barrier more than 2 ft beyond the edge of the required shoulder width should be evaluated. This assessment must include a comparison of the additional costs of all work such as benching, borrow, or grading needed to construct the flat slopes required to install barrier on the embankment, against the reduced cost of installation and maintenance of the lesser amount of barrier which will be required by locating it farther from the roadway. This assessment should also consider the location's accident history and the area's maintenance records regarding the repair of nuisance impacts.

49-4.01(02) Shoulder Section

On an INDOT route, the outside shoulder should be paved to the face of the barrier if such face is located 14 ft or less from the edge of the travel lane. On a local-public-agency route, the shoulder section at the barrier location may be designed as follows.

1. Where the face of the barrier is less than 2 ft from the outside edge of the paved shoulder, the shoulder should be paved to the face of the barrier.

2. Where the face of the barrier is 2 ft or more from the outside edge of the paved shoulder, the width of the paved shoulder may remain the same as in the sections without a barrier.

49-4.01(03) Barrier Deflection

If the width between the front face of a barrier in its correct location and the front face of an isolated hazard is less than the dynamic deflection width shown in Figure 49-4A, Barrier Deflections, the barrier's post spacing should be reduced to obtain a dynamic deflection width that is less than the width between the front face of the barrier in its correct location and the front face of the isolated hazard. If this is not practical, either the hazard or the barrier should be moved to provide adequate deflection width. A concrete barrier does not deflect.

The deflection widths for nonstandard guardrail type B are provided so that an existing installation can be analyzed to determine whether or not the existing deflection width is sufficient.

49-4.01(04) Shoulder- or Embankment-Slope Shielding Limits

The adjacent shoulder slope or embankment slope in front of a semi-rigid roadside barrier should desirably be 10:1 or flatter. Where site conditions dictate, a steeper such slope, though not steeper than 6:1, may be provided.

49-4.01(05) Barrier at Curb

A curb in front of a barrier may cause an errant vehicle to vault over, break through, or impact the barrier. However, there has been little research on which to recommend curb geometry or placement in the vicinity of a barrier. For this reason, the best practice is to avoid using a curb in the vicinity of a barrier. If a curb is essential for drainage, its effect can be minimized by using a maximum curb height of 4 in. and placing it so that the face of the curb is at or behind the face of the barrier.

In an urban situation, the barrier-curb combination should be offset at least the shy-line distance from the edge of the travel lane. This offset may either be continuous (curb with or without barrier) or variable as shown in Figure 49-4B, Barrier Placement at Curb. A continuous offset should be used if there are numerous separate runs of barrier along a curb to provide a uniform curb-line offset.

Where a barrier is to be installed in the vicinity of an existing curb, the curb should be removed unless the barrier can be placed as discussed above.

49-4.01(06) Lateral Placement for Large Drainage Structure on New Alignment, Excluding 3R Project

A large drainage structure is defined as that with a clear span of at least 66 in., as measured parallel to the roadway centerline, or a three-sided structure.

It is desirable to perpetuate as much of the clear zone as practical through a structure location. Where sufficient right-of-way will be acquired to provide the required clear-zone width, a barrier system described in Section 49-5.05 may be installed near the clear-zone limits. This is to shield the structure ends which are located within the clear zone, thus maintaining most of the clear zone required over the structure. However, where such barrier system is utilized near the edge of the clear zone, it should not be connected to another existing or proposed barrier that is located nearer to the pavement.

49-4.01(07) Lateral Placement for Large Drainage Structure on Existing Alignment, or 3R Project on New Alignment

Right-of-way may not be sufficient to perpetuate the clear-zone width through the structure location. The barrier should be installed at an offset of up to 2 ft from the edge of shoulder.

49-4.02 Barrier Length of Need

Figure 49-4C, Barrier Length of Need, illustrates the total length of need of a barrier, which is based on the equation as follows:

$$L_{TOT} = L_{ADV} + L_{HAZ} + L_{OPP} \qquad \text{[Equation 49-4.1]}$$

Where:

L_{ADV} = The length of need in advance of the hazard

L_{HAZ} = The length of the hazard itself

L_{OPP} = The length of the trailing end or length needed to protect traffic in opposing lanes.

49-4.02(01) Length of Need in Advance of Hazard for Adjacent Traffic [Rev. Sept. 2011]

Figure 49-4D, Barrier Length of Need in Advance of Hazard, illustrates the variables in the layout of an approach barrier to shield an area of concern for adjacent traffic. A roadside barrier should be installed parallel to the roadway. However, a flared installation may be appropriate where the barrier's end is buried in the backslope. Figure 49-4E, Design Elements for Barrier Length of Need, shows the runout length, L_R, and shy line offset, L_S, as a function of design year AADT and design speed. Figure 49-4F, Barrier Flare Rates, provides the flare rate, *a*:*b*, relative to the shy line. The shy line offset is defined as the distance beyond which a roadside obstacle will not be perceived as a threat by a driver. The barrier should be placed beyond the shy line offset. For a 3R project, it should be placed as described in Section 55-5.04(02).

The following procedures are used to determine the barrier length of need.

1. <u>Graphical Solution, Tangent or Inside Horizontal Curve</u>. One method of determining the length of need is to scale the barrier layout directly on the plan sheets as shown on Figure 49-4G, Barrier Layout, Bridge Approach. First, the runout length, L_R, is selected from Figure 49-4E. Then, the lateral distance to be protected is determined by calculating the clear-zone width, L_C, and comparing it to the lateral distance from the edge of travel lane to the outside edge of the hazard, L_H. The lesser of L_C or L_H is used to calculate the length of need, though a wider area may be chosen to be protected. Next, the runout length, L_R, and the lateral distance to be protected are scaled on the drawing along the edge of the travel lane, and a line is drawn between the lateral point farthest from the edge of the travel lane and the end of the runout length farthest from the hazard. This line simulates the vehicular runout path. To shield the hazard, the barrier installation must intersect this line. The barrier may be either flared or parallel to the roadway as dictated by site conditions.

2. <u>Graphical Solution, Outside Horizontal Curve</u>. For a length-of-need determination for the outside of a horizontal curve, the graphical solution should be used. The barrier length of need is determined by scaling its intercept with the tangential runout path of an encroaching vehicle rather than using the approach runout length, L_R. This is illustrated in Figure 49-4H, Barrier Layout, Fixed Object on Horizontal Curve. However, if the runout length measured along the edge of the driving lane is shorter than the distance to the tangential runout path intercept, the shorter distance should be used.

3. <u>Mathematical Solution, Tangent Section Only</u>. The required length of need may be
1 calculated using the formulas as follows:

For a flared barrier installation:

$$X = \frac{L_H + \left(\frac{b}{a}\right)(L_1) - L_2}{\left(\frac{b}{a}\right) + \left(\frac{L_H}{L_R}\right)} \qquad \text{[Equation 49-4.2]}$$

$$Y = L_H - \frac{(L_H)}{(L_R)}(X)$$ [Equation 49-4.3]

For a parallel barrier installation:

$$X = \frac{L_R(L_H - L_2)}{(L_H)}$$ [Equation 49-4.4]

Where:

X = length of need in advance of the hazard

Y = lateral offset to beginning of length of need on a flared barrier installation

Other variables are defined in Figure 49-4D, Barrier Length of Need in Advance of Hazard.

4. Minimum Length of Barrier. The minimum guardrail length required in advance of a hazard should be as shown in Figure 49-4E(1).

5. Guardrail Configuration at Approach to Bridge or Support. See the following figures to determine the guardrail configuration and minimum pay length for each situation listed below.

49-4E(2)	Guardrail Configuration for Outside-Shoulder Approach to Bridge
49-4E(3)	Guardrail Configuration for Median-Shoulder Approach to Bridge
49-4E(4)	Guardrail Configuration for Bridge Support Inside Clear Zone, Two-Way Roadway, Single Overhead Structure
49-4E(5)	Guardrail Configuration for Bridge Support Inside Clear Zone, Two-Way Roadway, Twin Overhead Structures
49-4E(6)	Guardrail Configuration for Bridge Support Inside Clear Zone, One-Way Roadway, Single Overhead Structure, Outside Shoulder
49-4E(7)	Guardrail Configuration for Bridge Support Inside Clear Zone, One-Way Roadway, Twin Overhead Structure, Outside Shoulder
49-4E(8)	Guardrail Configuration for Bridge Support Inside Clear Zone, One-Way Roadway, Single Overhead Structures, Median Shoulder
49-4E(9)	Guardrail Configuration for Bridge Support Inside Clear Zone, One-Way Roadway, Twin Overhead Structures, Median Shoulder
49-4E(10)	Guardrail Pay Length for Approach to Bridge Support

The L_{ET} portion of a guardrail end treatment type OS or MS, shown on Figures 49-4E(2) through 49-4E(9), should be considered as part of the guardrail length of need as described in Section 49-8.01(04) item 2.

49-4.02(02) Length of Need for Opposing Traffic

Figure 49-4 I, Barrier Length Beyond Hazard, 2-Lane Roadway, illustrates the layout variables of an approach barrier for opposing traffic. The length of need and the end of the barrier are determined in the same manner as for adjacent traffic, but all lateral dimensions are measured from the edge of the travel lane of the opposing traffic (e.g., from the centerline for a 2-lane roadway). For a 2-way divided roadway, the edge of the travel lane for the opposing traffic should be the edge of the driving lane on the median side. If a barrier is necessary to protect traffic in the opposing lanes, the minimum length of need is determined as follows:

1. If the design speed is 50 mph or higher, the required length in advance of the hazard for opposing traffic will be the greater of the calculated length or 100 ft.

2. If the design speed is 45 mph or lower, the required length of guardrail in advance of the hazard for opposing traffic will be the greater of the calculated length or 50 ft.

There are three ranges of clear-zone width, L_C, to be considered for an approach barrier for opposing traffic. In analyzing these situations, the type of treatment should be determined for a barrier or hazard where the barrier or hazard is just outside the clear zone. These ranges are as follows:

1. If the barrier is beyond the appropriate clear zone, no additional barrier is required. However, a crashworthy end treatment should be considered based upon AADT, distance outside the clear zone, and roadway geometrics.

2. If the barrier is within the appropriate clear zone but the hazard is beyond it, no additional barrier is required, but a crashworthy end treatment should be used.

3. If the hazard extends well beyond the appropriate clear zone (e.g., a river), the designer may choose to shield only that portion which lies within the clear zone, by setting L_H equal to L_C.

49-4.02(03) Length of Need Beyond Hazard for Divided Highway

Figure 49-4J, Barrier Length Beyond Hazard, Divided Highway, illustrates the procedure for determining the length of need beyond the hazard on a divided highway.

A gap of less than approximately 200 ft between barrier installations should be avoided, particularly if the cost of the additional barrier is about the same as the cost to install two separate end treatments, and access behind the barrier for maintenance or other purposes is not required. See the AASHTO *Roadside Design Guide*.

> ** *PRACTICE POINTER* **
>
> Barrier limits should be shown on the
> Plan and Profile sheets and in the Guardrail Table.

49-4.02(04) Length of Need at Outside-Shoulder Bridge Support [Rev. Sept. 2011]

Pier-protection barrier length for the right shoulder of a divided highway, or for both shoulders of a 2-lane, 2-way highway are based on the clear zone and the lateral location of the pier end relative to the clear zone. Depending on the lateral locations of the pier and the barrier, the barrier should either be fastened to the end of the pier or placed in front of the pier. The location and attachment is discussed below.

The additional barrier length required to protect another hazard in the area of the structure, such as a slopewall, bridge cone, or drainage structure under the slopewall, is computed separately.

If the conditions described below require calculations to determine the pier-protection barrier length, the calculation should consider all hazards adjacent to the pier end. These requirements apply to a pier for a single overhead structure, or twin (side-by-side) overhead structures spanning a 2-lane, 2-way roadway or divided highway, or tandem (end-to-end) overhead structures spanning a divided highway. The required length of pier-protection barrier is determined in accordance with the following:

1. <u>Support Located ≤ 16 ft from Edge of Travel Lane</u>. The support-shielding barrier is to be attached to the upstream traffic end of the support. The minimum required barrier length is shown in Figure 49-4E(1), Minimum Guardrail Length Required in Advance of Hazard.

   The length of need should be calculated using the equations shown in Section 49-4.02 and the clear-zone values from Figure 49-2A, Clear-Zone Width for New Construction or Reconstruction. The calculated length should be rounded up to the nearer whole multiple of 6.25 ft.

   If the support end is located inside the clear zone and the design speed ≥ 50 mph, the amount of barrier required should be the greater of the calculated rounded length or 100 ft. If the support end is located inside the clear zone and the design speed ≤ 45 mph, the amount of barrier required should be the greater of the calculated rounded length or 50 ft.

2. <u>Support Located > 16 ft from the Edge of Travel Lane</u>. The barrier length required in advance of the support is determined in the same manner as that required for each extended

hazard along the roadway. The support-protection barrier should be located between the support and the edge of travel lane and as far away from the edge of travel lane as feasible.

The lateral extent of the support foundation will dictate how close the barrier's posts can be driven to the support face. The barrier should be located such that the clearances from its face to the support face ≥ 4.25 ft and the clearance from its face to the pavement side edge of the support foundation ≥ 1.75 ft. These clearances are needed to permit the barrier to deflect upon impact without impacting either the support face or the foundation and to permit the driving of the post. If the clearance from the barrier face to the support face < 4.25 ft, the post spacing must be reduced in accordance with Figure 49-4A, Barrier Deflections. If the clearance from the barrier face to the support face < 2.75 ft, or the clearance from the barrier face to the pavement-side edge of the support foundation < 1.75 ft, the barrier should be installed in accordance with Item 1.

The required barrier length is shown in Figure 49-4K, Length-of-Need Requirement for Support Protection, and is described in Item 1 above. The barrier length along the face of the outside shoulder support or frame bent on a divided roadway should be sufficient to continuously cover the full length of the support plus 25 ft. For twin (in-line) supports, this length should also include the gap between the supports.

49-4.03 Example Length-of-Need Calculations

* * * * * * * * * *

Example 49-4.1

Given: Divided-highway structure over stream
Design speed = 65 mph
AADT = 7000
Foreslope = 4:1

Problem: Determine the length of the barrier needed on the shoulder side for the approaching end of the structure.

Solution: See Figure 49-4L, Barrier Length of Need, Structure-Approach Example 49-4.1

1. From Figure 49-3A, Transverse Slopes, determine clear-zone width, CZ = 46 ft.

2. From Figure 49-4E, Design Elements for Barrier Length of Need, determine runout length, L_R = 460 ft.

3. To find the point of *CZ*, first determine the hazard. In this situation, it is the stream. An errant vehicle must be protected from it.

4. To establish the point of *CZ*, first determine if the clear zone extends outside the right of way. If it does, the right-of-way line becomes the point of *CZ*, and where it crosses the top of the bank of the stream it becomes the point of *CZ*.

5. From the point of *CZ*, draw a line perpendicular to the edge of the travel lane and call this point E_P.

6. From point E_P, scale off distance L_R along the travel lane edge and call this point E_R.

7. From point E_R, to the point of *CZ*, draw a line.

8. Draw a line along the face of barrier parallel to the centerline from the bridge railing to where it crosses the line between E_R and the point of *CZ*. This is the barrier length of need for this particular bridge approach.

* * * * * * * *

Example 49-4.2

Given: 2-lane highway with high fill
Design speed = 60 mph
AADT = 7000
Right shoulder width = 10 ft
Slope in high fill area = 2.5:1
Slope at toe of fill = flat
Tangent
Level Conditions

Problem: Determine the length of barrier needed to protect the fill slope.

Solution: See Figure 49-4M, Barrier Length of Need, Fill-Slope Example 49-4.2

1. Determine clear-zone width, *CZ*, from Figure 49-2A. *CZ* = 30 ft based on flat slope at toe of fill. Therefore, adjusted *CZ* = 30 - 10 shoulder; or 20 ft at toe of slope.

2. Determine runout length from Figure 49-4E; L_R = 425 ft.

3. From Figure 49-3B(60), Barrier Warrant for Embankment, 2-lane, 2-Way Roadway, 60 mph, determine the location where the barrier should start. Interpolating between the 6000 AADT and the 12,000 AADT lines, the fill height = 8.9 ft.

4. At the point where the fill is 8.9 ft high, scale the L_R distance to point E_R.

5. From point E_R to point of *CZ*, draw a line.

6. Draw a line along the face of barrier parallel to centerline from the point where the fill height is 8.9 ft to where it crosses the line, between E_R and the point of *CZ*. This is the length of need required to shield the driver from the fill height.

8. The trailing end of a barrier run is determined in a similar manner, however, *CZ* is measured from the near edge of the opposing travel lane; see Section 49-4.02(02).

* * * * * * * *

<u>Example 49-4.3</u>

Given: Divided highway with large box culvert within clear zone that cannot be extended (under fill).
Design speed = 65 mph
AADT = 7000
Foreslope = 5:1

Problem: Determine the length of barrier needed to protect the driver from the culvert end.

Solution: See Figure 49-4N, Barrier Length of Need, Box-Culvert Example 49-4.3

1. Determine clear-zone width from Figure 49-2A; *CZ* = 38 ft.

2. Determine runout length from Figure 49-4E; L_R = 475 ft.

3. Using the end of the wing on the approaching-traffic side of the box culvert, draw a line perpendicular to the edge of the travel lane from the point of *CZ* through the end of the wing to the edge of the travel lane and call this point E_P.

4. From point E_P, scale along the travel lane the distance L_R and call this point E_R.

5. From point E_R to point of *CZ*, draw a line.

6. Draw a line along the face of barrier parallel to centerline from point E_P to where it crosses the line, between E_R and the point of *CZ*. This is the length of need on the approaching-traffic side.

7. The trailing end of a barrier run for the protection of the box culvert should be extended far enough to protect an errant vehicle from any hazard (for this example, a paved side ditch type F) when leaving the roadway at a 25-deg angle and missing the end of the barrier. Once this point has been established, add an additional 50 ft to establish the strength of the guardrail run.

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49-5.0 ROADSIDE-BARRIER APPLICATIONS

The basic parameter for roadside-barrier selection is the National Cooperative Highway Research Program *Report 350* (NCHRP 350) Test Level (TL) required at the site. This is a function of the following:

1. highway design speed;
2. adjusted construction-year traffic volume;
3. barrier offset;
4. highway geometry (grades, horizontal curvature);
5. height of bridge deck where applicable; and
6. type of land use below bridge deck, where applicable.

This Section provides the detailed methodology for determining the Test Level selection for each roadside barrier type. The methodology has been adapted from the AASHTO publication *Guide Specifications for Bridge Railings*. The *Guide Specifications* methodology is based on a benefit-cost analysis which considers occupant safety, vehicular types, highway conditions, and costs. The overall objective is to match each barrier's Test Level (and therefore costs) to site needs. Because of the similarities between the potential safety hazards from penetrating a roadside barrier, INDOT also applies this methodology to the Test Level selection for a median or shoulder barrier.

The NCHRP 350 Test Levels for roadside barriers used by the Department are described by the crash-test criteria shown in Figure 49-5A, NCHRP 350 Test-Level Crash-Test Criteria. Passage of a given crash test consists of a 75-ft length of a given device's ability to contain and redirect the test vehicle such that, after impact and stopping, the vehicle has remained upright and is pointing in its original direction in its original traffic lane.

A roadside barrier used on an INDOT-maintained route should be at minimum TL-3. A TL-4 or TL-5 barrier should be used where warranted as described below.

49-5.01 Roadside-Barrier Types

The specific types of roadside barriers for each Test Level are described below. Figure 49-4A, Barrier Deflections, provides the deflection distances for these barriers based on post spacing. The desired distance from the face of a roadside barrier to the shoulder breakpoint is 3 ft (1'-5" of barrier width plus 1'-7" behind it). In a restricted condition, this may be reduced to 1.5 ft.

49-5.01(01) TL-3 Barriers

1. W-Beam Guardrail with Posts Spaced at 6'-3". This barrier is used where the clearance between the guardrail face and the fixed object being shielded is 4.25 ft or greater.

2. W-Beam Guardrail with Posts Spaced at 3'-1½". This barrier is used where the clearance between the guardrail face and the fixed object being shielded is at least 3.25 ft but less than 4.25 ft.

3. W-Beam Guardrail with Posts Spaced at 1'-6¾". This barrier is used where the clearance between the guardrail face and the fixed object being shielded is at least 2.75 ft but less than 3.25 ft.

4. Nested W-Beam Guardrail. This configuration is used at a large drainage structure as described in Section 49-5.06. Nested guardrail at the post spacing listed above is also a TL-3 barrier.

5. Double-Faced W-Beam Guardrail with Posts Spaced at 6'-3". This barrier is used on a divided roadway as a median-side bridge-approach guardrail to one of the bridge structures in a set of twins.

6. High-Tension Cable-Barrier System (CBS). A CBS is a flexible median barrier with a larger lateral deflection during a vehicle impact than a semi-flexible barrier such as a double-faced W-beam or thrie-beam guardrail. A TL-3 CBS, if warranted, should be specified for a non-Interstate route. Design criteria are provided in Section 49-5.01(04).

49-5.01(02) TL-4 Barriers

1. Concrete Barrier, Shape F, Common Height. This barrier should be considered on the roadside to shield a rigid object where no deflection distance is available.

This barrier is used on an urban freeway where a barrier is required. If a rigid object is not continuous (e.g., bridge support), a half-section barrier may be used. To provide the necessary lateral support, backfill should be provided behind the half-section barrier, or the barrier should be tied to a concrete surface with reinforcing steel at its base. If this is not practical, a full-section barrier should be used.

2. Thrie-Beam Guardrail with Posts Spaced at 6'-3". If a deflection distance of at least 3 ft is available, this barrier should be considered in one of the following situations.

   a. New Facility, Location within the Limits of a Horizontal Curve with a Radius of 1435 ft or Less. The conditions which must be satisfied are as follows:

      (1) a barrier is warranted;
      (2) design speed is 50 mph or higher; and
      (3) design-year AADT is equal to or greater than 10,000.

   b. New Facility, Location on Horizontal Curve with Radius of Greater Than 1435 ft or on Tangent Roadway. The conditions which must be satisfied are as follows:

      (1) a barrier is warranted; and
      (2) design-year AADT is equal to or greater than 100,000.

   c. 3R or 4R Project, Location within the Limits of a Horizontal Curve with a Radius of 1435 ft or Less. The conditions which must be satisfied are as follows:

      (1) guardrail is in place and must be moved transversely to accommodate lanes or shoulders widened to 3R or 4R standards or horizontal curve improved to 3R or 4R standards, and such guardrail is still warranted;
      (2) design speed is 50 mph or higher; and
      (3) design-year AADT is equal to or greater than 10,000.

   d. 3R or 4R Project, Location on Horizontal Curve with Radius of Greater Than 1435 ft or on Tangent Roadway. The conditions which must be satisfied are as follows:

      (1) guardrail is in place and must be moved transversely to accommodate lanes or shoulders widened to 3R or 4R standards or horizontal curve improved to 3R or 4R standards, and such guardrail is still warranted; and
      (2) design-year AADT is equal to or greater than 100,000.

   e. Partial 3R Project. The conditions which must be satisfied are as follows:

(1) guardrail is currently in place;
(2) guardrail is still warranted; and
(3) a run of guardrail has been damaged, or gets impacted, on average, two or more times per year.

Guardrail impacts should be determined from the reported accident data (for the most recent available 3-year period) provided by the Planning Division's Office of Roadway Safety and Mobility. This information may be unavailable or may not indicate an average of at least two impacts per year. If so, the appropriate operations or maintenance personnel should be contacted for information which may reveal a history of an average of two or more impacts per year.

Each existing guardrail run of 300 ft or shorter which has been damaged, or gets impacted, on average, twice per year should be replaced with thrie-beam guardrail. An undamaged portion of at least 500 ft or longer of an existing W-beam run should be left in place. An undamaged portion of an existing W-beam run of less than 500 ft between high-impact areas should be replaced with thrie-beam guardrail.

f. Large Cross-Drainage Structure. Nested thrie-beam guardrail should be used at a large cross-drainage structure where nested guardrail is required, but a TL-4 device is warranted. Details for such thrie-beam configuration have not yet been developed as INDOT *Standard Drawings*.

Thrie-beam guardrail should be used instead of W-beam guardrail where a curb and sidewalk approach a bridge railing.

Thrie-beam guardrail should not be used for approaching a curved guardrail end treatment at a drive radius.

3. <u>High-Tension Cable-Barrier System (CBS)</u>. This is the same type of system as described in Section 49-5.01(01) item 6. A TL-4 CBS, if warranted, should be specified for an Interstate route. Design criteria are provided in Section 49-5.01(04).

49-5.01(03) TL-5 Barrier

The only TL-5 barrier used by the Department is the concrete barrier, shape F, truck height. This barrier may be used on the approach to a bridge, where warranted, to contain a large truck which can depart from the roadway, resulting in a high risk of loss of life or severe injury to a pedestrian or a person in a vehicle on a crossroad or a parallel road.

The TL-5 barrier should be used on the approach to a bridge where all of the following conditions exist.

1. The warrants for a TL-5 concrete bridge railing have been satisfied. See Section 61-6.01.

2. The mainline or ramp has a radius of 1435 ft or less.

3. The design-year AADT of the crossroad or parallel roadway below, which is within 120 ft of the edge of the overhead travel lane, is equal or greater than 7,500.

4. The physical characteristics of the roadside are such that an errant truck crashing through a TL-3 or TL-4 barrier can be expected to reach the crossroad, parallel roadway, or other high-occupancy land use area below.

For an existing facility, accident data should be obtained and analyzed. If an adverse truck-accident history is found, consideration should be given to installing the TL-5 barrier if the listed warrants are not satisfied.

Consideration should also be given to installing a TL-5 concrete barrier on each bridge approach of a new facility where motorist expectations are violated such as where a steep downgrade or long tangent section in advance of a curve over a crossroad will be constructed.

The length of need for a TL-5 barrier or TL-3 guardrail before and beyond the bridge is determined from the length-of-need equations for roadside barrier (see Section 49-4.02). The length of the TL-5 barrier should be based on the barrier length of need or the tangent runout path, whichever is less. Where a roadside barrier is warranted beyond the TL-5 concrete barrier, the additional barrier should be TL-3. Where the TL-5 approach barrier is used, it must be tapered down to the common height. Additional TL-3 guardrail beyond the concrete barrier must include a proper guardrail transition.

49-5.01(04) High-Tension Cable-Barrier System (CBS) Design Criteria

This positive-protection device should be considered in the median of a high-speed roadway where fatal median-crossover crashes have been reported or are anticipated.

1. <u>Warrants</u>. The lateral deflection of a CBS is 6.6 ft to 9.2 ft. A CBS may be used in a median of at least 36 ft width if the barrier is located close to the center of the median. It should not be located in a ditch bottom or flow line, so as to avoid potential drainage problems.

See the INDOT *Standard Drawings* for information on locating a CBS in a median which includes a bridge support, existing concrete barrier or guardrail, impact attenuator, or other safety hardware.

2. Advantages.

   a. A CBS can be installed in an existing median with a minimum of site work as one of the most cost-effective choices of median barrier.

      The cost of a CBS is almost the same as that of double-faced W-beam guardrail. Compared to double-faced W-beam guardrail, the repairs to a CBS are relatively simple, faster, and should not require driving posts or replacing rails.

   b. Vehicle containment and redirection are effective over a wide range of vehicle sizes and installation conditions. Deceleration forces upon vehicle occupants are low.

   c. A vehicle impact results in less damage to the vehicle and barrier, and results in less injury to vehicle occupants. The cable often remains at the proper height after an impact that damages several posts. A CBS can sustain multiple impacts and still remain effective.

   d. The posts are installed in sleeves in the ground to facilitate removal and replacement.

   e. Its open design does not generate drifting of sand or snow on or alongside the roadway.

   f. Once maintenance crews have developed the skills to rapidly repair a CBS, maintenance costs can be reduced.

3. Disadvantages.

   a. A comparatively long length of CBS is non-functional, and is therefore in need of repair following a vehicle impact.

   b. A large clear area is needed behind the barrier to accommodate the design lateral deflection distance.

   c. A CBS has reduced effectiveness on the inside of a horizontal curve.

   d. There is little installation tolerance in obtaining the specified barrier height.

e. Maintenance is often required.

4. <u>Design Considerations</u>.

a. Deflection. A CBS redirects an impacting vehicle after sufficient tension is developed in the cable, with the posts in the impact area offering only slight resistance. A deflection distance of 10 ft should be provided. The clearance between the cable and the opposing traffic's median edge of travel lane should be at least 10 ft.

The use of a CBS where it is likely to be impacted frequently, such as on the outside of a sharp horizontal curve, is not recommended.

b. Slope Requirement. A CBS should not be constructed on a slope steeper than 6:1. The approach should be relatively flat, without a curb or a ditch.

c. Transverse Location in Median. The post offset from the centerline of a median V ditch should desirably be at least 8 ft, or minimally within 1 ft of the centerline. The post offset from the edge of a median flat-ditch bottom should desirably be at least 8 ft or minimally within 1 ft of the ditch line. The post offset from the edge of paved shoulder should desirably be at least 12 ft to avoid nuisance impacts. The desirable conditions described above require a minimum median width of 48 to 52 ft for proper placement of a CBS assuming that the paved shoulder and flat-bottom ditch widths are each 4 ft.

d. Line Post and Anchor Foundations. Each end of a CBS run must be anchored. The designer should initially prepare a layout plan and request a geotechnical investigation of soil conditions for approximate locations of the safety terminals and representative locations of the intermediate line-post foundations. The geotechnical-investigation findings should be incorporated into the contract documents. End-anchor and line-post-foundation sizes are determined by soil classification, condition, temperature extremes, etc.

e. Line-Post-Foundation Size. The foundation for an intermediate line post should have a minimum depth of 3.5 ft and a minimum diameter of 14 in., with the foundation top flush with the ground level.

f. CBS Run Length. The recommended minimum run length is 1000 ft. The recommended maximum run length is 10,000 ft between anchors.

The number of median crossovers for emergency vehicles should correspond to that required with a concrete or thrie-beam median barrier.

g. Clearance to Rigid Obstacle. The lateral clearance to a rigid obstacle such as a bridge support, sign support, utility pole, tree, etc., should be 10 ft.

h. Placing CBS in the Vicinity of Another Barrier. If the side slopes are not steeper than 6:1 and another barrier is parallel to the roadway, the CBS can be tapered on a 50:1 or flatter taper. The end terminal should be placed behind the other barrier. A minimum lateral clearance of 10 ft from the end treatment of the parallel barrier is recommended. If the other barrier is flared, the CBS may be connected to W-beam or thrie-beam guardrail using an attachment to the guardrail end terminal that is available from the manufacturer.

i. Placing CBS in Vicinity of Inlet or Dike. If a drainage inlet, dike, etc., is encountered and cannot be adjusted to the proper grade, the CBS alignment should be gradually transitioned around it to ensure that the correct cable height above the ground line will be maintained. The horizontal transition should be on a taper of 50:1 or flatter.

j. CBS at Crossover. For a CBS termination at a median crossover, the CBS end terminal (end anchor) should be located beyond the tangent points of the crossover, preferably 3 to 5 ft from the tangent point.

k. Changing Offset of CBS in a Median from Being Closer to One Roadway to Being Closer to the Opposing-Traffic Roadway. If a CBS requires a change of lateral offset, the end anchors of the CBS should be overlapped for the minimum distance between the anchors in each direction as described below. The minimum distance for the anchor located at the incoming end should be at least the runout length, L_R, used for calculating the guardrail length of need. An overlap distance of 500 ft should be used for a median width up to 60 ft, a design speed of 70 mph, and AADT > 6000. For the anchor located at the outgoing end, the minimum overlap distance should be two times the anchor length. Changing the lateral offset of a CBS at the anchor located at the outgoing end is the preferable method.

l. Locating the End Anchor of CBS in the Vicinity of Impact Attenuator. If a CBS is terminated in the vicinity of an impact attenuator, the entire end-anchor length should be located at the distance shown on the INDOT *Standard Drawings* behind and clear of the concrete attenuator pad.

49-5.02 Existing Non-NCHRP 350 Guardrail to Remain in Place

Existing non-NCHRP 350 guardrail may be retained, subject to the following conditions.

1. A W-beam back-up plate is required at each W-beam-to-blockout connection where the W-beam element units are not lapped.

2. The height of guardrail should be a minimum of 2.25 ft with a maximum height of 2.5 ft as measured from the top of the W-beam to the ground surface at the face of rail.

3. A rubrail must also be used, including that for a guardrail run with a radius of 50 ft or less.

4. The flat-plate washers should be eliminated from under the head of the bolt holding the W-beam to the blockout except where washers are needed to transmit the forces in the W-beam to the anchor posts to obtain end anchorage. For example, if both ends of a guardrail run have positive anchorage at a bridge support or through a guardrail end treatment, all of the flat-plate washers should be eliminated except those in the transition. However, if the guardrail run ends without a positive connection, anchorage will have to be achieved through the last 5 posts and the washers must be left on these posts.

5. It is considered safer for an errant vehicle to traverse an embankment slope as steep as 3:1 at any height, than it is for the vehicle to impact a traffic barrier which can shield that slope (see Section 49-3.02). Therefore, on a reconstruction project, it may be necessary to remove portions of existing guardrail to be in accordance with to the concept that guardrail should be provided only where clearly warranted. However, on a slope steeper than 4:1, the clear runout area shown in Figure 49-2F, Clear-Zone Application for Non-Recoverable Fill Slope, must be provided at the toe of slope.

49-5.03 Roadside Barrier Requirement at Rock Cut

Where a barrier is required to shield a rock cut, a concrete shape F median barrier as described in Section 49-6.02(02) should be placed.

49-5.04 Roadside-Barrier Requirements at Bridge Pier

A pier located within the clear zone should be protected with guardrail. A pier located within 16 ft of the edge of the travel lane should be protected with a guardrail transition attached to the pier and the required length of guardrail. A pier located beyond 16 ft but within the clear zone should be shielded with either a guardrail transition attached to the pier and the required length of guardrail, or a run of guardrail placed in front of the pier, as determined on the field check (see Section 49-3.06).

See Section 49-8.02 for guardrail-transition information. Where the run of guardrail is placed in front of the pier, the offset between the face of rail and the edge of the travel lane should be made as large as practical. The clearance between the back of the guardrail posts and the pier should be checked to satisfy the guardrail-deflection criteria. Figure 49-3N, Treatment at Existing Bridge Cone with Shoulder Pier, provides typical details for shoulder-pier protection.

Where the offset distance between the face of pier and the edge of the travel lane is less than the minimum required usable-shoulder width, a design exception will be required for the shoulder width, though the pier is protected with guardrail. A design exception will not be required if the face of pier is located beyond the minimum required usable shoulder width, and the guardrail transition projects into the shoulder area.

The methods of treatment at an existing pier or bridge cone described above and the details shown on Figures 49-3L and 49-3M provide satisfactory methods of treatment. Because actual field conditions are variable, each location should be investigated at the field check to determine if alternative solutions may be more acceptable.

49-5.05 W-Beam Guardrail Over Large Drainage Structure Under Low Fill

A large drainage structure is defined as that with a clear span of at least 66 in., as measured parallel to the roadway centerline, or a three-sided structure. For such structure ends within the clear zone which are costly to extend and whose end sections cannot be made traversable, shielding with guardrail should be provided to protect an errant motorist from colliding with a structure end. If the structure end is outside the clear zone, guardrail should be placed to protect the errant motorist from the structure end.

If there is inadequate cover over the structure to support the guardrail posts, it will be necessary to use the details for guardrail installation over a low-fill structure as shown in the INDOT *Standard Drawings*. For this situation, full embedment of the guardrail posts is often impractical. The locations of the types of standard or modified posts are to be used should be shown on the plans.

Steel or concrete bridge railing in accordance with NCHRP 350 criteria also may be required over a low-fill structure where modified guardrail posts cannot be utilized. An appropriate guardrail-to-bridge-railing transition should be used.

The nested-guardrail configuration shown in the INDOT *Standard Drawings* should be used where there is inadequate cover for driving full-length guardrail posts. The configuration may be used within a longer run of W-beam guardrail, or may be used alone, depending on the length of guardrail need. This configuration has been crash tested in accordance with NCHRP 350 requirements, and approved for use by the FHWA on the National Highway System.

The configuration may only be used as one complete 100-ft unit. The number of modified posts should be determined, if they are required, to determine the pay quantity. The end-treatment requirements should also be determined.

The length of need for guardrail in advance of the structure or area of concern should be determined as described in Section 49-4.02. If nested W-beam guardrail is used over the structure and is not sufficient for the calculated length of need, additional non-nested W-beam guardrail should be provided to satisfy the length-of-need requirement preceding the nested W-beam guardrail installation as shown on the INDOT *Standard Drawings.* If there is a need for non-nested W-beam guardrail beyond the nested W-beam guardrail installation, the non-nested W-beam guardrail (minimum length 25 ft) should be connected to the outgoing end of the nested W-beam guardrail installation in lieu of the cable-terminal anchor system.

At an installation of guardrail for a large drainage structure on a 4R project constructed on new alignment, the shoulder should not be paved to the face of the guardrail. The standard width of stabilized shoulder should be specified.

Where W-beam guardrail is used to shield a structure, the following procedure should be used for each combination of overall structure width, W (ft), and depth of cover, C (ft), over the structure. The overall structure width of a large drainage structure is defined as the width out-to-out of structure parallel to the roadway centerline for a skewed or perpendicular structure.

49-5.05(01) Longitudinal Guardrail Placement

1. $\underline{W \le 24 \text{ and } C < 4}$. Use nested guardrail including a 25-ft span over the structure as shown on the INDOT *Standard Drawings.*

2. $\underline{24 < W \le 60 \text{ and } 1.5 \le C < 4}$. Use nested guardrail including a 25-ft span over the structure, and modified posts for the nested guardrail adjacent to the 25-ft span as shown on the INDOT *Standard Drawings*. The modified posts should be inserted into steel tubes, which are embedded into concrete bases. The concrete post bases should not be attached to the structure. The modified posts with concrete bases should only be used over the structure.

3. $\underline{W \text{ Not Limited and } 4 \le C < 5}$. Use TL-3 W-beam guardrail with 6-ft length posts at 6.25-ft spacing over the structure, and 7-ft length posts at 6.25-ft spacing preceding and beyond the structure.

4. $\underline{W \text{ Not Limited and } C \ge 5}$. Use TL-3 W-beam guardrail with 7-ft length posts at 6.25-ft spacing.

49-5.05(02) Cable-Terminal Anchor System

The cable-terminal anchor system may be used at the outgoing end of a W-beam guardrail run that is not exposed to oncoming traffic. It may be used as the equivalent of the W-beam anchorage guardrail ordinarily required 25 ft beyond the length of need, where space limitations do not permit placement of such a guardrail run.

49-5.05(03) Grading Requirements

Grading requirements for a structure carrying a rural divided highway on new alignment with a design speed of 70 mph are shown on the INDOT *Standard Drawings*. For a different design speed, a similar grading configuration should be designed using appropriate design criteria and dimensions.

Grading requirements for a structure carrying a highway on existing alignment without regard to design speed are also shown on the INDOT *Standard Drawings* for grading requirements at guardrail end treatment.

Guardrail length of need should be based on the clear-zone width.

49-5.06 Guardrail at Curb

If 2 ft of embankment (back of guardrail post to shoulder break point) cannot be provided behind a guardrail at a curb, nested guardrail should be used. Therefore, the guardrail post must be driven immediately behind the back of curb.

49-6.0 MEDIAN BARRIER

49-6.01 Median-Barrier Warrants

A median barrier should be used on a freeway or expressway where the design speed is 50 mph or higher, and median crossings are at least 1 mi apart. If breaks in the median barrier will, on average, be less than 1 mile apart, a median barrier should not be installed because of the larger number of barrier end treatments required. The hazards created by the end treatments are greater than the benefits derived from using a median barrier.

Figure 49-6A, Median-Barrier Warrants, provides the warranting criteria for median barrier on a freeway or other divided highway which has a relatively flat, unobstructed median. As indicated in

Figure 49-6A, a median barrier is warranted for combinations of 20-year projected AADT and median width that appear within the crosshatched area. At a low 20-year projected AADT, the probability of a vehicle crossing the median is relatively low. For a relatively wide median, the probability of a vehicle crossing the median is relatively low. These conditions are indicated by the shaded area under the curve. For a 20-year projected AADT less than 20,000 and a median width below the warranting curve, and for a median width 30 ft and below the warranting curve, median-barrier use is optional.

49-6.02 Median-Barrier Types

49-6.02(01) TL-3 Barrier

A double-faced W-beam guardrail system should be considered where median-barrier use is identified as optional as described in Section 49-6.01.

49-6.02(02) TL-4 Barriers

1. Concrete Barrier, Shape F, Common Height of 33 in. This barrier is used in a paved median of 36-ft width or narrower on a non-freeway. This barrier should be used where the impact frequencies are less than those described in Item 2.b. below, as this is a rigid system which will negligibly deflect upon impact.

   A modified concrete barrier may be necessary where the median barrier must accommodate a fixed object in the median (e.g., bridge pier, sign support). For details, see the INDOT *Standard Drawings*.

2. Double-Faced Thrie-Beam Guardrail with Posts Spaced at 6'-3". A median barrier must have been determined to be warranted as described in Section 49-6.01. Double-faced thrie-beam guardrail should be considered for an unpaved median where the minimum distance from the front face of the guardrail to edge of the paved shoulder is 12 ft. The designer should ascertain that the placement of guardrail posts does not interfere with sewer pipes, drainage structures, underdrains, etc.

   This barrier should be considered where a median barrier has been determined to still be warranted, and the following criteria are satisfied.

   a. New Facility. A median barrier is warranted as indicated by Figure 49-6A.

   b. Impact Frequency Where No Barrier Currently Exists. Impact data should be researched and applied as follows:

(1) there is an average of 0.50 cross-median crashes per mile per year; or

(2) there is an average of 0.11 fatal crashes per mile per year.

c. Impact Frequency Where W-Beam Guardrail Currently Exists. Researched impact data indicate that a particular run of guardrail has been impacted two or more times per year.

49-6.02(03) TL-5 Barrier

The only TL-5 barrier is the concrete barrier, shape F, truck height of 45 in. It should be used on a freeway as indicated in Figure 49-6A, Median-Barrier Warrants.

The following procedure should be used to determine if a truck-height median barrier is warranted on an expressway.

1. Determine adjustment factors K_g and K_c from Figure 49-6B, Grade Traffic Adjustment Factor, K_g, and Curvature Traffic Adjustment Factor, K_c. Use $K_s = 0.7$.

2. Calculate the adjusted construction-year AADT by multiplying the construction-year AADT shown on the plans (total for both directions) by the three adjustment factors and dividing by 1000 as shown below.

$$\textit{Adjusted construction-year AADT} = \frac{\textit{(construction-year AADT shown on plans)}(K_g)(K_c)(K_s)}{1000}.$$

3. Enter the Figures 49-6D series, Median-Barrier or Bridge-Railing Test-Level Selection for the appropriate design speed, for the type of roadway on which the work is located.

4. Locate the line in the figure that corresponds to the site conditions (% Trk and Edge of Travel Lane to Front Face Barrier, L_2).

5. Locate the adjusted construction-year AADT range, T, on the table.

6. If the calculated adjusted AADT value from Step 2 exceeds the T range from the figure from Step 5, a TL-5 railing or barrier is warranted. If the adjusted AADT is less, a lower Test Level railing or barrier is warranted.

7. If a TL-5 median barrier is warranted, it should be used between logical termini, such as two bridge piers.

The minimum length of need for a TL-5 concrete barrier in a median can be determined as discussed in Section 49-4.02(03). Other logical points of termination that should be considered include bridge pier or parapet, median crossover, or the beginning or end of project location.

This barrier may be warranted where there is a high volume of truck traffic, above deep water, on a high-occupancy land use area, on a high fill, across a deep ravine, or for a combination of these.

* * * * * * * *

49-6.03 Example for Determining Median-Barrier Test Level on an Expressway

Example 49-6.1 See Figure 49-6E, Truck-Height Concrete-Median-Barrier Example 49-6.1.

Given: 6-lane divided highway
Design speed = 70 mph
Construction-year AADT = 8,000 vpd
Percent trucks = 10%
Median width = 24 ft
Median-barrier offset = 11 ft
Horizontal curvature = tangent
Grade = 3% eastbound, -3% westbound

Problem: Determine whether a TL-4 or TL-5 concrete median barrier is appropriate.

Solution: Eastbound traffic, L_2 = 11 ft:

From Figure 49-6B, Grade Traffic-Adjustment Factor, K_g, and Curvature Traffic-Adjustment Factor, K_c, $K_g = 1.0$ and $K_c = 1.0$.
From Figure 49-6C, Traffic-Adjustment Factor, K_s, Deck Height and Under-Structure Shoulder Height Conditions, $K_s = 0.7$.

$$\text{Adjusted construction-year AADT} = \frac{(8{,}000)(1.0)(1.0)(0.7)}{1000} = 5.6$$

From Figure 49-6D(70), Median-Barrier or Bridge-Railing Test-Level Selection, Design Speed 70 mph, for % Trk $10 \le \% < 15$, $7 < L_2 \le 12$, and highway type as Divided, the appropriate T range is $2.6 < T \le 27.0$.

The value of 21.0 is within this range; therefore, a TL-4 median barrier may be used, and a TL-5 barrier is not required.

Westbound traffic, L_2 = 11 ft:

From Figure 49-4J, $K_g = 1.25$ and $K_c = 1.0$.
From Figure 49-4K, $K_s = 0.7$.

Adjusted construction-year AADT = $\frac{(8{,}000)(1.25)(1.0)(0/7)}{1000} = 7.0$

From Figure 49-6D(70), for % Trk $10 \leq \% < 15$, $7 < L_2 \leq 12$, and highway type as divided, the appropriate T range is $2.6 < T \leq 27.0$.

The value of 26.25 is within this range; therefore, a TL-4 median barrier may be used, and a TL-5 barrier is not required.

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49-6.04 Median-Barrier Design

49-6.04(01) Median Slopes

The slope in front of a median barrier should be 20:1 or flatter. Where a median barrier is warranted, it should be placed such that its effectiveness is not diminished by the severity of the median slopes. This may result in the placement of a median barrier along either or both inside shoulders instead of a single barrier along the center of the median.

49-6.04(02) Superelevated Section

Where a median barrier is located on the high side of a superelevated section, its vertical axis of symmetry should be at 90 deg to the pavement surface. On the low side of a curve, the axis of symmetry can be either vertical, or at 90 deg to the pavement surface. See Section 43-3.08 for more information on superelevation development with a median barrier.

49-6.04(03) Barrier-Mounted Obstacle

If a truck or bus impacts a median barrier, their high center of gravity may result in a vehicular roll angle which may result in the truck or bus impacting an obstacle on top of the barrier (e.g., a luminaire support). If practical, such an obstacle should be moved to the outside, or additional distance should be provided between the barrier and obstacle (e.g., a bridge pier).

49-6.04(04) Terminal Treatment

As with a roadside-barrier terminal, a median-barrier terminal also poses a potential roadside hazard for a run-off-the-road vehicle. Therefore, consideration must be given to the selection and placement of the terminal end. See Section 49-8.04 for information on impact attenuators.

49-6.04(05) Concrete-Barrier Height Transition

The truck-height concrete barrier should be tapered down to the common height where barriers of the two heights are connected as shown in the INDOT *Standard Drawings*. The transition should be sloped at 30:1 or flatter. This taper should be accomplished outside the area where the truck-height barrier is warranted. If the truck-height barrier does not connect to the common-height concrete barrier, the ends must be tapered down to the common height and terminated with an appropriate impact attenuator.

49-6.04(06) Horizontal Sight Distance

The use of a TL-4 or TL-5 barrier may limit stopping sight distance, SSD, on the inside of a horizontal curve. Therefore, the SSD should be checked on a horizontal curve to determine if the required SSD is available (see Section 43-4.0). If SSD requirements are not satisfied, the impacts of the reduced SSD on safety should be evaluated, and, if appropriate, a Level One design exception should be considered (see Section 40-8.0). If, for example, safety is significantly reduced, the TL-5 barrier may not be appropriate.

49-6.04(07) Intersection Sight Distance

The use of a truck-height median barrier may limit intersection sight distance, ISD. Therefore, the ISD should be checked as described in Section 46-10.03. If ISD requirements cannot be satisfied, the barrier height must be tapered to the common height as described in Section 49-6.04(05) as it approaches the portion of the barrier to be placed within the sight triangle. A common-height barrier and impact attenuator type SD may be extended into the sight triangle outside the limits of a public-road crossover or shoulder, and not beyond the stop line into the intersection. Consideration should be given to the ISD required for a vehicle turning right on a red signal indication after stopping.

49-6.04(08) Interchange Entrance Ramp

A motorist entering a freeway needs sufficient sight distance to locate gaps in the traffic stream in which to merge. The presence of a truck-height barrier can interfere with the sighting of an entering motorist. Therefore, the entrance ramp should be checked to ensure that adequate sight distance is available for the merge maneuver.

49-6.04(09) Median Barrier with Collector-Distributor Road

A concrete barrier may be warranted between a highway mainline and a collector-distributor road. In this situation, a TL-4 concrete barrier should be used because of the importance of sight distance.

49-6.04(10) Temporary Opening in Barrier

A temporary opening may be affected by using a gate device. Such opening may be used to route traffic around an emergency scene. An emergency opening may be required to route traffic around an emergency scene such that the roadway must be temporarily closed. For this situation, a proprietary device may be used to provide a temporary opening. It may be used in conjunction with a concrete median barrier to provide a temporary opening in the barrier for emergency vehicles or to temporarily reroute traffic. The device is opened and closed by means of an electronic control mechanism that can be manually overridden during a power failure.

49-6.05 Glare Screen

Headlight glare from opposing traffic can be bothersome and distracting. A glare screen can be used in combination with a median barrier to eliminate the problem. Specific warrants have not yet been adopted for the use of a glare screen. The typical application, however, is on an urban freeway with a narrow median and high traffic volume. Another application is between on/off ramps at an interchange where the two ramps adjoin each other. Here, the sharp radius or curvature and the narrow separation may make headlight glare bothersome. The use of a glare screen should be considered at either of these sites. A key element warranting its use is the number of public complaints received regarding glare for a particular highway section.

The following design criteria should be evaluated for a glare screen.

1. Cutoff Angle. A glare screen should be designed for a cutoff angle of 20 deg. This is the angle between the median centerline and the line of sight between two vehicles traveling in opposite directions. See Figure 49-6F, Cutoff Angle for Glare Screen. The glare screen

should be designed to block the headlights of oncoming vehicles up to the 20-deg cutoff angle. On a horizontal curve, the design cutoff angle should be increased to allow for the effect of curvature on headlight direction. The criterion is as follows:

$$\text{Cutoff Angle (deg)} = 20 + \frac{5731}{R}$$

Where R = horizontal radius (ft).

2. Horizontal Sight Distance. A glare screen may reduce the available horizontal sight distance. For a curve to the left, the middle ordinate must be checked to determine if adequate stopping sight distance will be available. See Section 43-4.0.

3. Sag Vertical Curve. In determining the necessary glare-screen height, the effect of sag vertical curvature need not be considered.

4. Height of Eye. The driver's eye height is 3.5 ft.

5. Glare-Screen Height. To determine the appropriate height of the glare screen, NCHRP *Synthesis 66, Glare Screen Guidelines* should be reviewed.

49-7.0 PIER OR FRAME-BENT COLLISION WALL

49-7.01 Application

A collision wall should be provided in new-construction or reconstruction work where the traffic face of an overhead-structure pier is not completely protected by guardrail or where there is a gap between adjacent piers that is not protected by guardrail.

For an overhead-structure frame bent (i.e., pier composed of columns), a collision wall should be constructed between the columns. For twin overhead structures, a collision wall should be constructed between the twin frame bents.

Such a wall is required for a shoulder-side or median-side pier or frame bent.

49-7.02 Design

The following provides the design criteria for a collision wall.

1. Wall Height and Thickness. The minimum height above the shoulder or ground surface should be 33 in. The minimum thickness should be equal to the thickness of the adjacent piers or bents. The height should be increased to match the height of the adjacent concrete median barrier.

2. Traffic-Face Geometry. The traffic-side face of the collision wall should be a vertical shape.

3. Footing Design. The footing should be 4 ft wide by 1 ft thick with the bottom 3 ft below the ground line. A longitudinal keyway is required at the top of the footing. The width of the keyway should be equal to one third the thickness of the wall, a minimum of 6 in., and with a depth of 3 in.

4. Reinforcing Steel. The longitudinal reinforcing steel should be #4 bars at 1'-0" spacing, the vertical reinforcing steel should be #5 bars at 1'-0" spacing, and the horizontal reinforcing steel at the top of the wall should be #4 bars at 1'-0" spacing.

5. Impact Attenuators for Median Pier or Frame Bent. An impact attenuator is required at each end of a median pier or frame bent for a single overhead structure. For twin overhead structures, an impact attenuator is required at the incoming end of the first structure and the outgoing end of the second structure on a divided highway.

6. Existing Collision Wall. An existing collision wall which is less than 33 in. in height above the shoulder or ground should be extended to 33 in. by grouting vertical #5 reinforcing bars at 1'-0" spacing into the top of the existing wall along both faces and pouring concrete to the necessary height.

7. Typical Collision-Wall Detail. Figure 49-7A illustrates typical details of a new collision wall.

49-8.0 GUARDRAIL END TREATMENTS, TRANSITIONS, AND IMPACT ATTENUATORS

49-8.01 Guardrail End Treatments (GRETs) and Usage

49-8.01(01) TL-3 Treatments

1. Type OS – Outside Shoulder. This type of GRET dissipates energy if hit head-on and has the ability to redirect an errant vehicle on one side only, where a backside impact is not anticipated. It is used with single-faced guardrail.

2. <u>Type MS – Median Shoulder</u>**.** This type of GRET dissipates energy if hit head-on and has the ability to redirect an errant vehicle on two sides, where a backside impact is anticipated. It is used with double-faced guardrail.

3. <u>Type II</u>. This type of GRET is used where a cut slope or backslope above the roadway grade is encountered along the roadside. The details for GRET type II are shown in the INDOT *Standard Drawings*. GRET type II is used to terminate single-faced guardrail in a backslope. This type redirects an errant vehicle on one side only. It is acceptable if the foreslope on the approach is 4:1 or flatter. It may be necessary to modify the details on the INDOT *Standard Drawings* to adapt to unique conditions. A deviation from the *Standard Drawings* should be shown on the plans. The design characteristics relative to guardrail design and embankment slopes shown in the INDOT *Standard Drawings* should be considered in the design.

   Where practical, it is desirable to bury the end of a guardrail run into the backslope. The factors to consider in burying guardrail in a backslope are proper guardrail flare, maintaining the proper height of the guardrail, providing proper shoulder, embankment, and approach slopes in front of the guardrail, and maintaining drainage.

   The design considerations to be evaluated in the selection of a GRET type II are as follows:

   a. A minimum 75-ft straight run of W-beam guardrail which may include a guardrail transition, is required preceding the area of concern (hazard).

   b. If this 75-ft guardrail run is not adequate, the guardrail run should be extended to shield the hazard.

   c. The cut slope or backslope should be located laterally approximately 6.5 ft minimum and 17 ft maximum from the face of guardrail, at the end of the 75-ft guardrail run. The backslope should be ascertained to extend parallel to the roadway for a sufficient distance to bury the end of the GRET type II, otherwise, a different type of GRET will be required.

   d. The total pay length of GRET type II includes both the WR-beam guardrail run and the guardrail-height taper to end anchorage. This buried-in-backslope guardrail end treatment is made up of the components as follows:

      (1) The first component is 25 ft of WR-beam guardrail at the specified ratio *a*:b, depending upon the design speed at the specific location.

(2) The length of the second component, which is also WR-beam guardrail, varies from 0 to 100 ft to fit field conditions at the specified ratio *a*:*b*, depending upon the design speed at the specific location.

(3) The third component is 37.5 ft of W-beam guardrail plus the steel-post anchor system at the specified ratio of 8:1.

e. For the buried-in-backslope guardrail system to be cost effective, the total length of the system should not extend approximately 150 ft beyond the guardrail length of need as determined in Section 49-4.02.

49-8.01(02) Non-NCHRP 350 Treatment

GRET type I is a treatment that may be used only on a local-public-agency route or on a local approach to an INDOT route, where the design-year AADT < 1000 regardless of the design speed. Double-faced GRET type I may be used in conjunction with a double-faced guardrail installation. GRET type I details are shown in the INDOT *Standard Drawings*. This guardrail end treatment type shall neither be used on the National Highway System nor an INDOT-maintained route.

This GRET should be flared. The embankment in the flared area should be sloped at a 20:1 rate. If the guardrail is on a taper, it is acceptable to continue the buried end on the same taper line without offsetting it further, provided the minimum 2-ft offset is obtained.

49-8.01(03) Design Considerations

The considerations which should be evaluated in the design of a GRET or guardrail transition are described below.

1. <u>Slopes</u>. All slopes in the area of a GRET should be graded in accordance with the INDOT *Standard Drawings*.

2. <u>Breakaway-Cable Terminal</u>. A breakaway-cable terminal end section should be removed and replaced with the NCHRP 350 GRET which is suitable for the location.

3. <u>Transition</u>. A guardrail transition to a bridge pier, bridge railing, etc., should be as shown on the INDOT *Standard Drawings*.

4. <u>Opening Near a Bridge</u>. A drive or a county road may intersect the highway a short distance from the end of a bridge. Providing an opening in the guardrail for such an approach should

be accomplished by using the curved W-beam guardrail terminal or connector system as shown on the INDOT *Standard Drawings*.

5. GRET Type OS or MS. This GRET should be installed in alignment with the guardrail if the guardrail run is on a tangent. For a curved guardrail run, the GRET should be constructed along a chord of the curve with the beginning and end of the GRET having the same offset from the edge of the travel lane (see Figure 49-8A, Guardrail End Treatment Type OS or MS for Curved Guardrail Run).

6. W-Beam Guardrail Buried in Backslope. Where practical, consideration should be given to burying the end of a guardrail run into the backslope. Further considerations include proper guardrail flare, maintaining full design height of guardrail, and providing proper drainage and approach-terrain details. In addition, the following should be considered.

    a. Flare Rate. The guardrail system should be flared away from the roadway at a rate not greater than 15:1 until the guardrail passes the clear zone or the center of the ditch, whichever is the greater distance. At that point, it can then be flared back at 8:1. The foreslope in front of the guardrail should be 20:1. A steeper slope, up to a maximum of 10:1, may be used if necessary to allow for ditch grading.

    b. Guardrail Height. The design height should be maintained across the slope to the point where the guardrail passes over the foreslope-backslope intercept. Where this is not practical and if the gap between the ground and the bottom of the W-beam rail is 1.75 ft or more, it will be necessary to add a W-beam rubrail. The rubrail should be added for 50 ft downstream and 25 ft upstream of the area where the gap exceeds the 1.25-ft normal height. The W-beam rubrail should be terminated behind the last post, similar to that shown for a guardrail transition type VH on the INDOT *Standard Drawings*.

    c. Anchors. The end of the guardrail buried in the backslope will be anchored with a W-beam steel post anchor system as shown on the INDOT *Standard Drawings*.

    d. Transitions. A foreslope transition zone will be needed to transition from the standard ditch cross-section in the cut section to the 10:1 desirable, 6:1 maximum, foreslope in front of the guardrail. The approach slope to the 20:1 cross slope in front of the guardrail should be a 30:1 maximum longitudinal slope relative to the roadway grade. The ground can then be warped from the standard ditch cross-section to the desired 10:1 foreslope in front of the guardrail. These conditions, if satisfied, should minimize the potential for a vehicle to vault over the guardrail or for wheels to snag on the guardrail.

e. Drainage. Where a ditch section providing the recommended guardrail approach terrain cannot be constructed without blocking flow in the ditch or where the resulting ditch grade is too slight, an acceptable inlet type and an outlet pipe will be required to carry the drainage under the guardrail. Where an inlet is not needed in the vicinity of the guardrail because of approach-terrain requirements, there may be a need for a drainage structure behind the guardrail in the fill section to prevent erosion.

7. Drive-Behind. If an errant vehicle penetrates the guardrail end treatment section, the motorist should be able to guide his or her vehicle down the slope without difficulty. Therefore, a minimum recovery area behind the barrier end treatment must be provided. This recovery area is shown in Figure 49-8B, Clear Recovery Area Behind Guardrail.

49-8.01(04) Design Procedure [Rev. Sept. 2011]

After the design of a roadside barrier is completed, including the determination of the barrier length of need and the appropriate railing transitions in accordance with Section 49-8.03, it is necessary to select the proper GRET.

In order to determine the appropriate GRET type, the following should be considered.

1. Relationship of GRET to Traffic. It must be determined if there will be traffic on one or both sides of the guardrail end treatment. The GRET may be located beyond the outside shoulder with traffic passing on one side only, or it may be in a median, gore, or other location where traffic passes on two sides. If all traffic will pass a GRET only on one side, the GRET will not require redirective capability on more than one side. If traffic will pass the GRET on two sides, it may be necessary for the GRET to be capable of redirecting errant vehicles from two sides.

    a. GRET for Single-Faced Guardrail. For this situation, the GRET must provide redirective capability only on the traffic side. GRET type OS or type II should be selected for this situation.

    b. GRET for Double-Faced Guardrail. For this situation, the GRET must provide redirective capabilities on both sides. GRET type MS should be selected for this situation.

2. Relationship Between GRET and Guardrail Length of Need. Some GRETs can function as typical guardrail as described below.

a. GRET Type OS. A 37.5-ft portion of the downstream end of a GRET type OS can function as typical guardrail. It therefore should be considered as part of the length of need in advance of the obstruction. Where GRET type OS is warranted, the pay length for the guardrail run is equal to the required length of need for the guardrail minus 37.5 ft.

b. GRET Type MS. A 12.5-ft portion of the downstream end of a GRET type MS can function as typical guardrail. It therefore should be considered as part of the length of need in advance of the obstruction. Where GRET type MS is warranted, the pay length for the guardrail run is equal to the required length of need for the guardrail minus 12.5 ft.

GRET type I or II cannot function as typical guardrail, so no portion of it should be considered as part of the guardrail length of need.

The reduced pay length should be reflected in the guardrail length shown on the plans.

49-8.02 Guardrail Transitions and Usage

49-8.02(01) TL-3 Transitions

1. Type WGB – W-beam, Guardrail to, Bridge railing transition. This transition type is used where the proximity of an intersecting road or drive prevents the proper installation of the guardrail transition type TGB described in Section 49-8.02(02). Where at least one transition type WGB is required at a bridge, all bridge-railing ends should use the transition type WGB.

2. Type GP – Guardrail to Pier. This transition is used to connect guardrail to a bridge pier or a frame bent.

49-8.02(02) TL-4 Transitions

1. Type TGB – Thrie-beam, Guardrail to, Bridge railing transition. – This is the preferred transition. It should not be used only where an intersecting road or drive prevents the placement of a properly designed system. To use the transition type TGB, there must be space to place at least 25 ft of roadside barrier between a curved W-beam guardrail connector terminal system or a curved W-beam guardrail system and the beginning of the transition.

2. Type WGT – W-beam, Guardrail to, Thrie-beam guardrail transition.

a. Outside Shoulder. A thrie-beam section must be transitioned to a W-beam section, and a guardrail end treatment type OS should be attached to the end of the W-beam section. This transition connector is guardrail transition type WGT. The details are shown on the INDOT *Standard Drawings*. The WGT guardrail transition must be used to bring the thrie-beam guardrail to the W-beam guardrail height for proper attachment of a guardrail end treatment.

b. Median-Side Shoulder. Where thrie-beam guardrail is terminated in a median, two WGT transitions with staggered posts as shown on the INDOT *Standard Drawings* must be provided unless a median pier or barrier wall, etc., is immediately adjacent. The two WGT guardrail transitions must be used to bring the double-faced thrie-beam guardrail to the double-faced W-beam guardrail height and width for proper attachment of a guardrail end treatment type MS.

49-8.02(03) Non-NCHRP 350 Transition

Type VH – Vertical Height adjustment – may be used to extend existing non-NCHRP 350 guardrail classes Bs, Ds, Es, or Hs if adding new TL-3 guardrail. This transition involves the vertical adjustment of the first 25 ft of existing guardrail adjacent to the new guardrail. The adjustment requires the posts in this 25-ft section to be driven deeper to compensate for the height difference between the two guardrail systems, and it also requires the proper termination of the rubrail. This transition is also used where a GRET type MS or OS is being connected to an old railing system. To properly specify the required version of this transition, the post spacing of the existing guardrail adjacent to the proposed extension must be known.

<u>49-8.03 Bridge-Railing Transitions</u>

See Section 61-6.0 for more information on the location and design of a bridge-railing transition and its complementary bridge railing.

49-8.03(01) TL-2 Transitions

A TL-2 transition should only be used on a non-INDOT-maintained route not on the National Highway System.

1. <u>Type TGS-1 – Transition, Guardrail, Side-mounted, 1 tube</u>. This transition is used with bridge railing type TS-1.

2. Type TPF-2 – Transition, Pedestrian-height, Flush with deck, 2 tubes. This transition is used with bridge railing type PF-2.

3. Type TPS-2 – Transition, Pedestrian-height on, Sidewalk, 2 tubes. This transition is used with bridge railing type PS-2.

4. Type TTX – Transition, TeXas 411 ornamental. This transition is used with bridge railing type TX.

49-8.03(02) TL-4 Transitions

A TL-4 transition should be used on an INDOT-maintained route or the NHS where a TL-5 railing and transition is not warranted.

1. Type TBC – Thrie-beam, Bridge approach, Common height. This transition is used with the common-height, shape F concrete bridge railing.

2. Type TPF-1 – Transition, Pedestrian-height, Flush with deck, 1 tube. This transition is used with bridge railing type PF-1.

3. Type TPS-1 – Transition, Pedestrian-height on, Sidewalk, 1 tube. This transition is used with bridge railing type PS-1.

4. Type TGT – Thrie-beam, Guardrail, Truck height. This transition is used with bridge railing type CF-1

5. Type TTT, Thrie-beam guardrail, Transition to, Thrie-beam bridge-railing transition. This transition connects a bridge-railing transition to the thrie-beam guardrail by providing a height-adjustment transition. The TTT transition details are shown on the INDOT *Standard Drawings*.

49-8.03(03) TL-5 Transition

Type TBT – Thrie-beam, Bridge approach, Truck height is used with concrete bridge railing, shape F, truck height, and with bridge railing type TR.

49-8.04 Impact Attenuators

49-8.04(01) Types

Impact-attenuator selection design is based on the appropriate Test Level for the design speed of the roadway under consideration.

The types of TL-2 or TL-3 impact attenuators are described as follows:

1. Type ED – Energy Dissipation. This is an energy dissipation device.

2. Type R1 – Redirective 1 side. This is an energy dissipation device that has redirective capability on one side.

3. Type R2 – Redirective 2 sides. This is an energy dissipation device that has redirective capability on two sides.

4. Type CR – Clearance Restriction. This is an energy dissipation device that has redirective capability on two sides. This type is used where there are lateral clearance restrictions that make installation and maintenance of the attenuator difficult.

   The expected or experienced crash frequency should be considered in attenuator type CR selection.

   Type CR1 should be specified unless conditions exist as described below.

   Type CR2 should only be specified for a location that has been documented for an existing alignment, or anticipated for a new alignment, by the appropriate district maintenance engineer, to have an impact frequency of 3 or more per year. A type CR2 unit is largely self-restoring after a typical impact, and has the ability to partially absorb additional impacts that can occur before the unit can be serviced.

   The designer should solicit input from the appropriate district maintenance engineer on which type of CR attenuator to specify. Use of a type CR2 attenuator must be authorized in writing by the maintenance engineer.

5. Type SD – vertical Sight Distance limitation. This is an energy dissipation device that has redirective capability on two sides. This type is used at an intersection where there can be sight distance limitations if a taller attenuator is used.

If the design speed is 45 mph or lower, the attenuator design should be in accordance with TL-2 criteria. A project with a design speed of 50 mph or higher will require an attenuator design which should be in accordance with TL-3 criteria. An attenuator shielding an obstruction located between roadway facilities with different design speeds (e.g., gore area) should be in accordance with the Test Level requirement for the higher design speed.

An impact attenuator type LS – Low Speed – is a low-speed energy dissipation device that has redirective capability on two sides. This type should be in accordance with TL-1 criteria only. Attenuator type LS should be selected for a design speed of 30 mph or lower. The type SD attenuator may also be used in this situation.

49-8.04(02) Design

After the design of a roadside barrier is performed in accordance with Section 49-5.0, it is necessary to determine whether there is an obstruction located within the clear zone that is not protected. An obstruction that can be protected by extending a proposed barrier a short distance should be protected in this manner. However, an impact attenuator should be utilized to protect an isolated obstruction.

Unless transitioned to a roadside barrier, the end of a truck-height bridge railing should be shielded with an appropriate impact attenuator. This applies whether the end is inside or outside the clear zone.

If an impact attenuator is required for a median barrier near an at-grade intersection, intersection sight distance should be checked as described in Sections 46-10.03 and 49-6.04(07). If sight distance is inadequate, an impact attenuator type SD should be placed to protect the median-barrier end.

Figure 49-8C, Impact-Attenuator Offsets, illustrates common impact-attenuator installations. The D1 dimension shown on the figure determines whether an attenuator is warranted and, if so, whether the attenuator requires redirective capability on the side adjacent to the traffic under consideration. The D2 dimension shown on the figure is used to determine whether the attenuator requires redirective capability on its backside.

For an obstruction in a gore or other similar area, the offset dimension from the edge of the obstruction face to the mainline outside travel lane edge must be compared to the similar measurement between the obstruction and the ramp inside travel lane edge. The smaller of the two offsets is defined to be D1 and the larger offset is considered to be D2.

The required attenuator-width designation is based on the width of the obstruction. The standard available widths are as follows.

1. <u>W1</u>. This attenuator width is required for an obstruction that is not more than 3 ft wide.

2. <u>W2</u>. This attenuator width is required for an obstruction that is more than 3 ft wide but less than or equal to 6 ft wide.

3. <u>W3</u>. This attenuator width is required for an obstruction that is more than 6 ft wide but less than or equal to 8 ft wide.

Impact attenuator type ED is limited to the W1 width only. A width requirement greater than that provided by width W1 will necessitate the selection of an impact attenuator type R1 or R2.

Impact attenuator type LS is limited to the W1 width only. A width requirement greater than that provided by width W1 will necessitate the selection of an impact attenuator type R2 or CR.

For the terminal end of a concrete median barrier, an impact attenuator type R1 or R2 is used.

For another impact-attenuator type, if the obstruction width is greater than 8 ft, the obstruction should be shielded with an attenuator specifically designed for that width, altered so the width is less than or equal to 8 ft, or moved to a location where shielding is not required.

Figure 49-8D, Impact-Attenuator Type Determination, illustrates the space requirements for each approved impact attenuator. For a roadway with a shoulder section, the attenuator footprint shown on the figure should not encroach onto the usable shoulder, as defined in Chapter Fifty-three, Fifty-four, or Fifty-five, as appropriate.

For a roadway with curbs, the attenuator footprint should not encroach onto the 1.5-ft appurtenance-free zone, as discussed in Section 49-2.03(04). If the roadway section includes a sidewalk, the attenuator footprint should not encroach upon the sidewalk to reduce the remaining sidewalk width to less than 4 ft. An impact attenuator should not be installed behind a curb. Where necessary for drainage, a sloping curb not higher than 4 in. may be used for at least a distance of L_R in advance of and alongside the attenuator. If the attenuator footprint violates the encroachment limits described above, the obstruction should be shielded with a roadside barrier, altered so the footprint encroachment is satisfactory, or moved to a location where shielding is not required. See Figure 49-8E, Impact-Attenuator Footprint Requirements.

49-8.04(03) Requirements at a Median Pier

The type of protection required for a pier or frame bent located in a median is determined by the configuration of the overhead structure. The possible overhead-structure configurations are single, twin (side-by-side), or tandem (in-line). The required pier protection is determined as follows and is summarized in Figure 49-8F, Pier-Protection Requirements.

1. Single Overhead-Structure Pier or Frame Bent. The protection required is based on the clearance from the face of the pier or frame bent to the median edge of the travel lane.

2. Twin (End-to-End) Overhead-Structure Piers or Frame Bents. The protection required is based on the clearance from the faces of the piers or frame bents to the median edge of the travel lane at the outermost ends of the piers or frame bents.

3. Tandem (In-Line) Overhead-Structure Pier or Frame Bent. Due to the bridge-cone location behind the median-side pier or frame bent for this type of overhead structure, the pier protection should be the same as that required for outside-shoulder location described in Section 49-3.06.

49-9.0 BRIDGE-RAILING END

49-9.01 Curved W-Beam Guardrail System

The curved W-beam guardrail system is composed of two subsystems. The first is the curved W-beam guardrail terminal system, which is used to terminate a guardrail run where the run is interrupted by a drive. The second subsystem is the curved W-beam guardrail connector system, which is used to connect guardrail located along a main roadway to guardrail or a guardrail end treatment located along an intersecting public-road approach. Each subsystem includes different types which can be specified based upon site conditions.

The area behind the curved W-beam guardrail system should be cleared of all fixed objects which constitute hazards as shown on the INDOT *Standard Drawings*.

49-9.02 Bridge-Railing-End Shielding [Rev. Sept. 2011]

The AASHTO *LRFD Bridge Design Specifications* requires that each bridge-railing end be shielded from direct collision by traffic. The type and extent of protection required should be determined based on the location of the bridge-railing end relative to the clear zone. The minimum extent of protection should be as shown in Figure 49-4E(1), Minimum Guardrail Length Required in Advance of Hazard. Conditions in an urban area can preclude the protection as shown in Figure 49-4E(1). See *LRFD Bridge Design Specifications* Article 13.7.1.2 and its Commentary for other options.

The required length of bridge-approach guardrail, including the guardrail transition, for both shoulders of a 2-lane, 2-way highway, or the outside shoulders of a divided highway, is based on the clear-zone requirement for the roadway and the design speed. The calculated length should be rounded up to the nearer whole multiple of 6.25 ft. The length shown in Figure 49-4E(1) is that required to shield the end of the bridge railing only and should be considered the minimum requirement. All hazards adjacent to the bridge-railing end should be considered where bridge-approach-guardrail length is to be determined.

49-9.03 Public Road Approach or Drive

Each public road approach or drive that prohibits the installation of the required bridge-approach guardrail and guardrail end treatment should be relocated or closed. Because this will not always be practical, each situation must be addressed individually, with emphasis placed on providing the maximum protection practical consistent with the restrictions.

The appropriate guardrail layout at, and in advance of, the public-road approach or drive is dictated by the control line, which is established by the clear zone and the guardrail runout length, L_R.

49-9.03(01) Public-Road Approach

Where a public road approach cannot be relocated, the appropriate curved W-beam guardrail system should be specified, in accordance with the INDOT *Standard Drawings* and the guidelines included herein. A minimum of 25 ft of W-beam guardrail should be provided between the guardrail transition type TGB and the curved W-beam guardrail system. Where this is not practical, a bridge railing transition type TBC and a guardrail transition type WGB should be specified instead of the type TGB, to connect the concrete bridge railing to the curved W-beam guardrail system.

A curved W-beam guardrail connector type 1 or type 2 should be used depending on the system radius required to come in contact with the approach radius. The following should be considered.

1. Curved W-Beam Guardrail Connector System, End Located At or Beyond the Control Line. Where the end of the curved W-beam guardrail connector system is at or beyond the control line, as shown in Figure 49-9B, Public-Road-Approach Application At or Beyond the Control Line, no additional guardrail is required along the public road approach. An appropriate guardrail end treatment should be used to attach to the end of the curved W-beam guardrail connector system. The area in advance of the guardrail, bounded by the edge of travel lane and the control line, must be traversable. The additional grading should be shown on the plans.

2. Curved W-Beam Guardrail Connector System, End Located Within the Control Line. Where the end of the curved W-beam guardrail connector system is within the control line, as shown in Figure 49-9C, Public-Road-Approach Application Within the Control Line, additional guardrail will be required from the end of the curved W-beam guardrail connector system to the control line, terminated with an appropriate guardrail end treatment.

3. Guardrail Requirements for Public-Road Approach. If additional guardrail is needed to satisfy the clear-zone requirements along a public-road approach, this guardrail should extend from the end of the curved W-beam guardrail connector system to the point of need along the public-road approach and be terminated with an appropriate guardrail end treatment.

49-9.03(02) Drive

Except as described below, a curved W-beam guardrail terminal system type 1 or type 4 should be used depending on the system radius required to come in contact with the drive radius. The following should be considered.

1. Type 5 Anchor Located At or Beyond the Control Line. Where the type 5 anchor of the curved W-beam guardrail terminal system, as shown in Figure 49-9D, Drive Application At or Beyond the Control Line, is at or is entirely beyond the control line, the bridge-approach guardrail should be terminated at that point. However, the area in advance of the guardrail, bounded by the edge of travel lane and the control line, must be traversable. The additional grading should also be shown on the plans.

2. Type 5 Anchor Located Partially or Entirely Within the Control Line. Where the type 5 anchor of the curved W-beam guardrail terminal system, as shown in Figure 49-9E, Drive Application Within the Control Line, is partially or entirely within the control line, the guardrail run should be continued on the other side of the drive to the point of need. This will require another curved W-beam guardrail terminal system along the other side of the drive, additional W-beam guardrail along the roadway shoulder in advance of the drive, and an appropriate guardrail end treatment. This advance guardrail should be extended from the end of the curved W-beam guardrail terminal to the point of need and then connected to the guardrail end treatment. However, if this guardrail length required in advance of the drive is less than 100 ft, the guardrail run and curved W-beam guardrail terminal system in advance of the drive will not be required. However, the area in advance of the guardrail, bounded by the edge of the travel lane and the control line, must be traversable. This additional grading should be shown on the plans.

3. Restricted Right of Way. Where the obtainable right of way is insufficient for use of the normal configuration, a modified version of the curved W-beam guardrail terminal system should be used. A modified version has shorter legs along the side of the drive and is designated as type 2, 3, 5, or 6, as shown in the INDOT *Standard Drawings*. Types 2 and 5 are 6.25 ft (one panel) shorter than the standard version. Types 3 and 6 are 12.5 ft (two panels) shorter than the standard version. The appropriate type should be chosen based on the system radius required to come in contact with the drive radius and the amount of

shortening required by the restricted right of way. The restrictions concerning the location of the type 5 anchor and the need for additional guardrail in advance of the drive are still applicable to this situation.

Examples of restricted right of way include avoidance of a wetland or other environmentally-sensitive area or a lawn. An example of an area where additional right of way should be purchased to avoid removing guardrail panels is agricultural land. For a 3R project, the criteria shown in Section 55-5.04(02) Item 5 should be considered. The guardrail run may be shortened or the guardrail terminal system may be eliminated.

49-9.04 Unfavorable Site Conditions

Site conditions will frequently be encountered which prohibit or restrict the use of these treatments. The necessary drive or approach relocation, additional right of way, and clearance for each fixed obstacle should be obtained to provide the suitable protection. If these efforts are not practical, a project-specific design may be necessary. The Production Management Division's Roadway Standards Team should be contacted for assistance.

49-9.05 Median-Shoulder Bridge-Approach Guardrail Length

The length of median-shoulder bridge-approach guardrail is based on the clear-zone requirements for the roadway. The entire length of the median-shoulder bridge-approach guardrail, exclusive of the bridge railing transition type TGB, is double faced. The required minimum length is shown in Figure 49-9F, Median Bridge-Approach Criteria. The flare and offset shown is the desired layout of the guardrail. The length of bridge-approach guardrail should be recomputed for site conditions other than those assumed and listed in Figure 49-9F.

49-10.0 GUIDE TO THE ROADSIDE COMPUTER PROGRAM

This Section supplements the information in AASHTO *Roadside Design Guide*, Appendix A, and in the README file of the ROADSIDE computer program. It provides more detailed information and guidance on the use of ROADSIDE and an expanded listing of recommended severity indices and an example of a sensitivity analysis.

49-10.01 Introduction

The program ROADSIDE is a useful tool for highway engineers making decisions for the design of roadsides and the placement of highway hardware. It aids the designer in selecting an alternative treatment which offers the greatest anticipated return for safety benefits for funds expended. ROADSIDE is the microcomputer version in the AASHTO *Roadside Design Guide*, Cost-Effectiveness Selection Procedure. The program is written in Quick Basic 4 and is not copyrighted. Thus, modifications to the program can be made if the user has an understanding of basic programming and the assembled language of the program.

49-10.01(01) Using ROADSIDE

With the computer turned on, insert the ROADSIDE disk into the CD drive. At the DOS prompt, change to the appropriate drive, type ROADSIDE and press Enter.

The program then reads the data files containing the lateral extent of encroachment probabilities and displays a note on the screen to that effect.

The Basic Input Data Screen (Figure 49-10A) and global values are then shown, with an inquiry to the user regarding the value to be used. If no changes to the basic input data are desired, type N (no) and press Enter. The severity index versus cost relationship is displayed next for the user's information. Press Enter to continue.

The Variable Input Data Screen (Figure 49-10B) is the last screen displayed. All data entry occurs on this screen. To enter data, type the appropriate line number from the left-hand margin and press Enter. A new screen will then be displayed showing the current value and asking the user to enter the new value for the field in question. All calculations are automatically made as the user inputs values for each variable. Whenever an input variable is changed, all calculations using that variable are automatically made and the new results are displayed.

The Command Menu at the bottom of the Variable Input Data screen identifies the function keys listed below that are used in ROADSIDE.

49-10.01(02) Function Keys

The following function keys are used in the program:

1. <u>Function Key 1</u>. This key will print a copy of the Variable Input Data screen and the resultant computations. The printout contains some information that does not appear on the computer screen. The computer screen was modified so all data entry can be made on a single screen.

2. <u>Function Key 2</u>. This key will store the problem variables and basic input data.

3. <u>Function Key 3</u>. This key will retrieve a previously stored problem. The user will be given two or three options. If the problem was stored with the original default values, the user may have the problem recalled to the screen using the default data or using the basic input data values from the last problem shown on the screen (called the "current" values). If the problem was stored using altered values, then it may be recalled using those values ("dataset" values), using the "default" values, or using the basic input values that were used on the last problem shown on the screen ("current" values).

4. <u>Function Key 4</u>. This key will let the user access the HELP menu which contains detailed information on every aspect of ROADSIDE.

5. <u>Function Key 5</u>. This key will display, and allow the user to change, the basic input (global) values.

6. <u>Function Key 6</u>. This key will display the relationship between severity index and cost as derived from the accident costs included in the basic input values.

7. <u>Function Key 7</u>. This key will list all file names on the ROADSIDE disk.

8. <u>Function Key 8</u>. This key lists the percentage of accident types included for each severity index value.

9. <u>Function Key 9</u>. This key will, for computers with graphic display capability only, provide a sketch of the highway roadside, and hazard parameters. The "Print Screen" key will allow the user to obtain a hard copy of this sketch if a dot matrix printer is used. A "daisy wheel" will not print correctly.

10. <u>Function Key 10</u>. This key is used to exit the program. No data are stored via this function. Data should be stored using Function Key 2.

<u>49-10.02 Basic Input Data</u>

The first input screen (Figure 49-10A) shows all default values. While these numbers represent the best judgment of the program developers, the user of this program has the option to change any default value as deemed appropriate based on new data or on local conditions. If no changes are made in these variables, the program then prints out accident costs for each severity index based on the default accident costs by accident type.

The swath width is the effective width of an encroaching vehicle that is not tracking. Although this width naturally varies depending on vehicular length, width and yaw angle, a width of 12 feet is the default value used to represent a typical vehicle. The yaw angle, shown in Figure 49-10C, is defined as the angle between the direction the vehicle is traveling and the direction the vehicle is pointing. This value may be changed if desired, but it is considered both reasonable and representative for analysis purposes.

Accident costs are assigned to each of three categories of accidents — fatal, injury and property damage only (PDO). Injury and PDO accidents are further divided into different levels of severity. The default values in the program may be changed, but it is recommended that the default values be used for lack of more current information. Accident costs used in economic evaluations differ significantly between agencies. The default values in the model were selected as median values. Should they be changed, the values assigned to these, especially fatal accidents, will have a significant effect on the numerical values and the calculated cost-benefit ratios, but it will usually not change the relative ranking of the alternatives being considered. The effect of using one set of values over another can be assessed using a sensitivity analysis. This procedure is illustrated with the example problem where the same alternatives are analyzed using the default accident costs included in Figure 49-10A and with the FHWA-recommended costs from FHWA Technical Advisory T 7570.1.

49-10.03 Variable Input Data

The second input screen (Figure 49-10B) in the program includes specific roadway and roadside characteristics that must be entered by the user. The program contains Lateral Extent of Encroachment Probability tables for 40, 50, 60, and 70 mph, and adjustment coefficients for horizontal curvature and grade.

The following subsections describe each of the input data and explain how they are used in this program. Figure 49-10D is provided for quick reference.

49-10.03(01) Title

Each alternative or iteration should be assigned a unique title if it will be saved for later retrieval and comparison to other alternatives. When saving an alternative, a unique file name will also be required. The title and file name need not be the same.

49-10.03(02) Traffic Volume and Growth

Line	Input Data	Units
2	*Traffic Volume*	*two-way ADT*
	Growth Rate	*percent*

Enter the current daily 2-way traffic volume and an estimated annual growth rate. The traffic growth rate is entered as a percentage (0 to 10%). In the absence of other guidance, a traffic-growth rate of 2.0% is suggested.

The model assumes the characteristics of the highway facility are uninterrupted flow with no interaction among vehicles in the traffic stream. Once the traffic volume reaches capacity, the characteristics change to interrupted flow and the volume-encroachment relationship is no longer valid. Therefore, a default value limits maximum traffic volume to 10,000 vehicles per lane per day. A volume higher than 10,000 is reduced to 10,000 vehicles per lane per day in the first year only. The program does not limit or omit a volume which may exceed 10,000 vehicles per lane per day during the remaining project life. ROADSIDE does not assign traffic to individual lanes on multi-lane highways. This is discussed in Section 49-10.03(03).

A divided-roadway facility will operate at uninterrupted flow except for peak hours. The 10,000 limit may be too low because the facility will operate at uninterrupted flow the majority of the time. A higher limit of 15,000 vehicles per lane per day may be used for a divided highway.

Traffic volume is a significant factor for determining user costs; therefore, using accurate volumes is important. The growth rate usually does not significantly affect the user and agency costs. A general rate readily available should be used because of this.

49-10.03(03) Roadway Type

Line	Input Data	Units
3	*Roadway Type*	*undivided (U), divided (D), one-way (O)*
	Lanes of Adjacent Traffic	*number of lanes*
	Width of Each Lane	*feet*

Enter the type of highway being analyzed. Three options exist — divided, undivided, and one-way. For undivided highways, encroachments on one side of the road by both adjacent and opposing traffic are calculated. Encroachments from the opposite direction are not computed on divided and one-way highways. The number of lanes of adjacent traffic and the width of each lane must also be entered. Adjacent traffic is defined as all lanes traveling in the same direction on the roadway next to the obstacle. A 2-lane undivided highway will have one adjacent lane of traffic whereas a 4-lane divided highway will have two adjacent lanes.

The obstacle can be located in the median or to the right of the traveled way. The model does not recognize whether the encroachments occur on the inside (median) or outside of the roadway. The user should treat the median as if it is a roadside. An analysis in the median may also require separate program runs so that encroachments are considered from both directions.

The total traffic volume is split equally between both directions of travel, except for one-way roadways or ramps. The directional volume is assigned to the lane closest to the obstacle. In actuality, there is a distribution of total traffic between the travel and passing lanes for a multi-lane highway. Most of the traffic in the travel lane will be an additional 12 feet from a hazard located in the median. Therefore, the number of encroachments may be overestimated for a median-side analysis, where the lane closest to the obstacle normally carries lighter traffic volume. An analysis more representative of the actual lane distribution could be obtained by running the program separately for each lane. Figure 49-10E can be used to select approximate lane distributions for 4- and 6-lane highways. With each program run, the only input variables that would change are traffic volume and the distance to the obstacle. An alternative method is to apply the appropriate factor in Figure 49-10F and Figure 49-10G; this provides the same answer as the sum of separate program runs.

49-10.03(04) Geometric Adjustment Factors

Line	Input Data	Units
4	*Roadway Curvature Adjustment*	*degrees*
	Roadway Grade Adjustment	*percent*

There are two geometric adjustment factors for the encroachment rate. These are listed below:

1. Roadway Curvature Factor. Curves to the right (for adjacent traffic) are assigned a (+) sign and can increase the basic encroachment rate by a factor of 2 (maximum) for curves of 6 degrees or sharper. Curves 3 degrees or flatter do not increase the basic rate.

   A curve to the left (for adjacent traffic) is assigned a (-) sign and can increase the basic encroachment rate by a factor of 4 (maximum) for curves of 6 degrees and sharper. A curve of 3 deg or flatter do not change the basic rate. ROADSIDE selects the appropriate factor when the degree of curvature is entered.

2. Roadway Grade Factor. Negative grade (downgrade) in the direction of adjacent traffic increases the basic encroachment rate by a factor of 2 for a 6% or steeper grade. A downgrade of 2% or less does not affect the basic rate. The appropriate factor is selected once the grade is entered by the program user.

   For example, a tangent highway section 1/3 mile in length with 6,000 AADT will have a calculated value of 1 encroachment for two years (1/3 mile x 3,000 AADT per direction x 0.0005 encroachment rate x 2 years = 1). This is neglecting opposite direction encroachments. If that highway section was on a 6-degree curve with a 6% grade, there would be 8 encroachments on the outside downhill curve [4 (curve factor) x 2 (grade factor) x 1 encroachment = 8] and 2 encroachments on the inside uphill curve [1 (curve factor) x 2 (grade factor) x 1 encroachment = 2].

49-10.03(05) Encroachment Rate

Using the data up to this point (lines 2, 3 and 4), the program automatically computes the total number of encroachments. An encroachment begins when a vehicle leaves the roadway (i.e., crosses the edge of the travel lane and/or moves onto the shoulder). The number of encroachments is shown for the total adjacent and opposing traffic (see Figure 49-10B). Adjustments are made for roadway characteristics (horizontal and vertical alignment) which will increase the number of encroachments.

The user adjustment factor allows the user to modify the basic rate if there are site specific conditions or an accident history that warrant a change. The user factor can be used to adjust the predicted number of encroachments with actual conditions or historical data.

As mentioned earlier, the user factor could be used to adjust for encroachments on multi-lane highways. This saves a step in running the program once versus several times for each lane. Figures 49-10F and 49-10G provide factors to use for analyzing either the median or outside of either a 4- or 6-lane highway.

49-10.03(06) Design Speed

Line	Input Data	Units
6	*Design Speed*	*miles per hour*

The design speed of the roadway is used to select a lateral-extent-of-encroachment probability curve. Curves for speeds of 40, 50, 60, and 70 mph are used in the program. For any input speed less than 40 mph, the 40-mph curve is used; the 50-mph curve is used for speeds between 40 and 50; the 60-mph curve is used for speeds between 50 and 60, and the 70-mph curve is used for speeds above 60 mph. These curves assume flat side slopes and underestimate the lateral extent of encroachment when slopes steeper than 10:1 exist. They may also overestimate the lateral distance a vehicle is likely to travel on a backslope. A design speed lower than the posted speed limit should not be used. At site specific locations, generally use speeds that closely approximate the actual or anticipated operating speed of the facility. At certain sites, such as some suburban highway sections with large peak hour volumes, the average operating speed may not accurately represent the design speed. In these cases, use the low-volume operating or running speed which represents the most likely condition for a single vehicle off roadway accident.

49-10.03(07) Hazard Definition

Line	Input Data	Units
7	*Hazard Offset from Driving Lane*	*A, feet*
	Hazard Length (parallel to road)	*L, feet*
	Hazard Width (perpendicular to road)	*W, feet*

ROADSIDE defines a roadside hazard as a rectangle that is laterally offset from the edge of the driving lane a distance of A feet, is L feet long in the direction of travel, and W feet wide. The hazard can be a bridge pier, a large box culvert inlet and channel, an embankment, or a traffic barrier designed to shield a roadside obstacle or non-traversable terrain feature.

Defining the area of concern for multiple obstacles can be difficult. The program should not be run several times for each obstacle and composite costs added. Such an analysis implies a degree of accuracy the model lacks. In some cases the hazard may be behind another hazard (i.e., trees behind traversable ditch, 3:1 slope with trees at bottom, etc). In some cases there may be multiple hazards (trees on slope, culvert outlet on slope, etc). In defining these hazards, a single program run is accurate enough. This will require the user to select a rectangle that includes all significant hazards, a procedure similar to defining an area of concern for barrier layout (page 5-32, 1988 AASHTO *Roadside Design Guide*). For varying or multiple offset distances, an average offset distance should be used. The severity index may also need to be adjusted to account for various combinations of hazards; see Section 49-10.03(09).

User costs are sensitive to the offset distance and length of obstacle. The closer to the roadway and the longer the obstacle, the bigger the chance for collision. Agency costs are also sensitive to obstacle length. The width of the obstacle does not significantly influence costs.

49-10.03(08) Collision Frequency

Using the data supplied up to this point (lines 2 through 7), the program calculates the collision frequency. Once you have defined an object and determined how far it is from the ETL, the number of vehicles which hit the object is automatically calculated. The expected number of collisions with the hazard each year is the summation of collisions into the side, corner and longitudinal face of the hazard by adjacent and (where applicable) opposite-direction traffic. The input screen shows the initial collision frequency (impacts per year) for the whole object and for each location on the hazard impacted (face, side and corner). The collision frequency over the life of the project is only shown on the output screen.

Collision frequency is basically an accident rate for the object's exposure, because the number of impacts is determined over the length of the object. For example, a 1,000-ft length of guardrail, 8 ft from the ETL on a 6,000 ADT 2-lane roadway, will have an estimated number of 0.22926 impacts for the first year. Over five years, this equates into 1 accident (0.22926 x 5 years) for that 1000-ft section of guardrail.

49-10.03(09) Severity Index

Line Input Data

9 *Severity Index for:*

upstream side of hazard (SU)
downstream side of hazard (SD)
upstream corner of hazard (CU)
downstream corner of hazard (CD)
longitudinal face of hazard (FACE)

To convert accidents to costs, a severity index (SI) must be assigned to impacts with the hazard. Essentially, assigning a SI to an object is determining the relative cost per accident. The relationship between severity index and the percent accident type is shown on page A-12 of the *RDG*. For example, assigning a SI of 5.0 for a tree is predicting that resulting impacts will be 8% fatalities, 77% injuries, 15% PDO. Taking each percentage by accident costs (e.g., 8% x $500,000, etc.), the predicted cost per accident is $56,535.

ROADSIDE has no capability to select an appropriate SI and is dependent upon the user for this information. The more severe an object (higher SI), the higher the associated accident costs are. Once a SI is assigned to an object, the program automatically computes the resultant accident costs.

Impacts into a given object may have different outcomes based on where the vehicle hits. Therefore, adjustments can be made for impacts into the side of the hazard, the upstream and downstream (for 2-way traffic) corners of the hazard, and the face of the hazard. These will be equal for point objects such as trees and utility poles. For barriers, the severity of the accident will be less for a face impact than for a side or corner hit.

Figures 49-10H through 49-10P have been developed to provide more information to the user. Accident data was not used to develop the table. To determine SI's from accident records would require detailed accident data for each roadside object or obstacle. Unfortunately, accident reports seldom contain all the information needed to identify the object or obstacle struck in detail. The SI is a relative value, rather than an absolute or discrete number. It does not represent an impact into a specific object at the selected design speed, but rather an average estimated impact speed, given the selected design speed. This means that for most features there will be many low-severity accidents included. A low-severity accident is one in which a vehicle is nearly stopped before reaching a feature, or strikes it such that its occupants are not seriously injured. That is why the numbers are generally lower than the values in the 1977 *Barrier Guide*, which represented the severity of crashes at 60 mph. The tables were developed by ranking each common object by speed (e.g., different types of guardrail, etc).

The severity indices shown on Figures 49-10H through 10P incorporate ranges for each obstacle. The range covers other performance factors beyond those considered in the model. The user should read the information when selecting a value within the range. The ranking was based on the anticipated performance and intuitive judgment from engineers with backgrounds in safety, design and research. Based on historical data of relative relationships (guardrail and slopes, guardrail and ditches, etc.), the common objects were then compared to one another and adjustments were made as deemed appropriate. Severity for the sides and corners are assumed to be the same values shown for the side. Both mean that the severity for the face, corner, and side impacts are the same. These objects have also been listed in the *RDG* Appendix A in order of ascending severity for each speed (40, 50, 60, and 70 mph).

There are many cases where different obstacles will appear within the clear area. Each will have its own relative severity index (e.g., a tree on a 3:1 slope, headwall and culvert opening, curb and guardrail, culvert opening and 4:1 slope). The severity table could not possibly provide a severity index for each situation. The combination of hazards adds more uncertainty as to the collision outcome. Adjustment to the severity index within the given range or even outside the range may be required.

The severity index is a very significant factor in determining user cost. Designers will need to use their best judgment in selecting a value. The sensitivity of different values should be analyzed for their impact on resulting costs. A sensitivity analysis over a range of values would be appropriate because of the variable's significance. In any case, the analyst should always apply the test of reasonableness to the output of ROADSIDE and be wary of using the results to compromise established safety practices or to justify costly or controversial new safety design practices or policies.

Actual accident history can be used to determine a cost per accident. One method for determining an average cost per accident is described in FHWA Technical Advisory T 7570.1, dated June 30, 1988. By using the SI - accident costs relationship, accident costs could be used to find a SI. As mentioned above in using actual data several gross assumptions need to be made, one of which is the model's prediction of collisions versus reported accidents. Not all collisions will result in an accident. Vehicles may drive away from an impact to a slope or guardrail. An adjustment based on a ratio of actual accidents to predicted collisions needs to be made on the SI. Additional information in this area is included in Appendix F in TRB Special Report 214.

49-10.03(10) Project Life and Discount Rate

Line Input Data Units

10	*Project Life*	*years*
	Discount Rate	*percent*

The project life of a roadside design is the useful life of the design and is an input value selected by the user. The discount rate is also a basic input to the economic analysis. Once these variables are selected, the program calculates the economic factors needed to complete the analysis. In the absence of other guidance, a discount rate of 4.0% is suggested.

The project life is the time period from construction to replacement of each alternative. This is also called the alternative's useful life and may have a significant effect on the analysis. There are many situations at a given location where alternatives will have different useful lives. For consistency it would be desirable to establish a common or national figure for useful lives for each alternative. Such values could not be applied at each situation because of the many uncertainties involved. It is recommended that the useful life be established for the analysis by using the best information available to an agency. Typically, 20 years is used; beyond 20 years the accuracy of the predictions is difficult to estimate. A sensitivity analysis can be used to compare different periods of time for a given location.

The discount rate usually is not a significant factor in the analysis. High rates favor future investments and low rates favor current investments. The discount rate is used to reduce various costs or benefits to their present worth or uniform annual costs so that the economics of different alternatives can be compared. If the discount rate is set equal to the real interest rate (interest minus inflation), reasonable values are in the order of 3 to 5 percent.

49-10.03(11) Highway Agency Costs

Line	Input Data	Units
11	*Installation Cost*	*dollars*
12	*Repair Cost (per accident)*	*dollars*
13	*Routine Maintenance Cost (per year)*	*dollars*
14	*Salvage Value*	*dollars*

The installation (construction), repair, maintenance and salvage value costs are the final basic inputs to the program. Once this information is provided, total present worth and annualized costs and highway agency present worth and annualized costs are computed. This is the output of the program, which enables the design engineer to make direct comparisons between several proposed alternative safety treatments.

Direct costs include construction, maintenance, repair and salvage. The most important of these costs is construction cost. Because this is a significant factor, the construction cost used in the analysis should be current and can be obtained from the latest *INDOT Catalog of Unit Price*

Averages for Roads - Bridges - Traffic. A sensitivity analysis comparing variations in cost may be desirable.

Routine repair costs for a number of different types of barriers, end treatments and crash cushions are shown in Figure 49-10Q. These should be used to estimate the repair costs for these items unless better information is available.

Due to subjectivity and difficulty of determining routine maintenance costs and salvage values, the user can typically assume these to be $0 (or zero).

49-10.04 Analysis Methods

The three common methods used to compare alternative proposals in an economic analysis are as follows:

1. comparison of present worth of costs;
2. comparison of equivalent uniform annual cost; and
3. benefit/cost ratio.

When properly applied and when the results are properly interpreted, each method will lead to the selection of the same project as being the most economically advantageous. Each alternative must be compared with the others to determine the best selection when more than two alternatives are being compared.

In the present worth method (PW), the objective is to compare the present worth of all cash flows for a selected time period. The alternative having the minimum present worth is normally the best selection. The present worth represents the sum which would be required in the base year to finance all future expenditures (agency and user's) during the project life. ROADSIDE automatically computes the total present worth for each alternative. The analysis period for which the present worth costs are calculated must be equal for all alternatives.

In the equivalent uniform annual cost method (EUAC), all alternatives are compared on the basis of their equivalent uniform annual cost. The alternative having the minimum total EUAC is most often the selection of choice. ROADSIDE automatically computes the EUAC for each alternative. Comparison of alternatives with different analysis periods can be made. This is assuming construction replacement costs are the same in the future.

The benefit/cost ratio method measures the ratio of expected benefits to cost. These costs are usually expressed as a EUAC. The B/C ratio method is an incremental solution; i.e., it compares the differences of a pair of alternatives. Usually alternatives which include a safety improvement are compared with existing conditions (i.e., do nothing). Benefits are the reduction in accident costs (accident costs for do nothing minus accident costs for the improvement). Costs for the B/C ratio would be agency costs for that improvement.

49-10.05 Sensitivity Analysis

There are many factors which influence traffic safety policies and the development of safety programs. Rational decision-making processes combined with a cost-effective analysis are of crucial importance in the choice between competing social and economic goals. The cost-effective selection procedures provide a basic tool to compare alternative roadside improvements at site-specific locations. It was intended for evaluating improvements to either reduce the chances of a crash (remove or relocate) or reduce the severity (retrofit or shield). The decision between doing nothing and safety improvements is another question. Existing policies and standards are the overriding force in this area. ROADSIDE provides a basic tool for comparing alternative improvement options at specific locations. However, it is a probability model and the ranking of options should be viewed as a relative ranking only. Furthermore, the program is extremely sensitive to the selection of a severity index and to the costs assigned to each general type of accident.

Sensitivity is the relative effect that a variable may have on the decision. The sensitivity of each input variable on the user and agency costs are summarized in Figure 49-10R. Use of the computer program makes it relatively easy to vary an input variable. It may be desirable to test the effects of variations of the significant input variables on the selection of an alternative.

49-10.06 Examples

These examples are from the Federal Highway Administration's August 1991 *SUPPLEMENTAL INFORMATION FOR USE WITH THE ROADSIDE COMPUTER PROGRAM.* The options considered in these examples may not always correspond to those required by INDOT policy.

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Example 49-10.1 Culvert and protruding headwall.

Use the example problem (provided in the AASHTO *RDG*, Appendix A) and check the effects of changing accident costs and severity.

Design options: Option 1 - do nothing
Option 2 - shield the culvert
Option 3 - extend the culvert
Option 4 - modify culvert inlet/outlet

Sensitivity Analysis:

1. See how a change in accident costs affects the outcome (*RDG* default values vs. FHWA T 7570.1 values)

   FHWA T 7570.1: Fatal accident = $ 1,500,000
   Injury = $39,000 - $12,000 - $6,000
   PDO = $2,000

2. See how changes in severity indices affect the outcome (*RDG* SI values vs. suggested SI values in this Section).

Summary:

1. <u>Accident Cost</u>. Annualized cost using *RDG* accident cost default values.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$2,060	$0	$2,060	n/a
Option 2	$858	$392	$1,250	3.1
Option 3	$225	$625	$850	2.9
Option 4	$591	$441	$1,032	3.3

Annualized Cost for FHWA T 7570.1 accident cost values.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$4,966	$0	$4,966	n/a
Option 2	$1,661	$392	$2,053	8.4
Option 3	$542	$625	$1,167	7.1
Option 4	$1,240	$441	$1,681	8.4

Discussion:

The sensitivity analysis shows that increasing the accident cost would increase the benefit-cost (B/C) ratio 2 to 3 times. The benefit (reduced accidents from existing condition - Option 1) increases for each option because of the higher relative accident cost. In most cases, using a higher accident cost will not change the order of which option has the highest B/C ratio, but the B/C ratio may change significantly for an object with a high severity index. The example problem shows Option 4 has the highest B/C ratio when using default accident values but, when the accident costs are increased, both Option 4 and Option 2 have the same B/C ratio. The two options in either case are close enough that there is no clear cut answer. In fact, if another analysis method is used, equivalent uniform annualized cost (EUAC), Option 3 is the best choice. The user should be aware that a change in any of the input variables may alter the order of which option has the best B/C ratio. In making a decision, the analyst should obtain more information about existing practices and constraints of each option. Selection of the best option should be based on results of the model, additional information and good engineering judgment.

2. Severity Indices. *RDG* SI values in example/modified SI values in this Section (using *RDG* default accident cost).

Impact Location	Option 1	Option 2	Option 3	Option 4
Upstream side	5.5/5.4	3.0/3.4	5.5/5.4	4.0/3.2
Downstream side	5.5/5.4	3.0/3.4	5.5/5.4	4.0/3.2
Upstream corner	6.0/5.5	3.0/3.4	6.0/5.4	4.0/3.2
Downstream corner	6.0/5.5	3.0/3.4	6.0/5.4	4.0/3.2
Face	4.8/4.2	2.7/3.2	4.8/4.2	4.0/3.2

SI Selection:

Option 1 - Side: high-range of culvert >3 feet
Corner: mid-range projecting headwall >10 inches
Face: high-range of vertical wall
Option 2 - Side and corner: low-range of BCT
Face: low-range W-beam guardrail
Option 3 - Side and corner: high-range of culvert >3 feet
Face: high-range of vertical wall
Option 4 - Side, corner and face: slightly higher than high range for a 4:1 slope (10-ft embankment)

Annualized cost using different severity indices (RDG accident cost values).

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$1,629	$0	$1,629	n/a
Option 2	$1,395	$392	$1,787	0.6

Option 3	\$167	\$625	\$792	2.3
Option 4	\$310	\$441	\$751	3.0

Discussion:

In changing from the *RDG* SI values to the modified SI values, the following changes occur — Option 2 (shield) drops from a B/C ratio of 3.1 to be less cost-effective than the do-nothing option, Option 3 (extend) drops from a B/C ratio of 2.9 to 2.3; Option 4 (modify opening) drops from a B/C ratio of 3.3 to 3.0. Option 4 has the lowest EUAC of \$751. Option 2 (barrier) has a larger exposure area than the existing conditions and, therefore, the calculated number of accidents will increase. Although the severity of the barrier is less than the existing culvert opening, the severity reduction is not enough to make the installing barrier cost-effective. If FHWA accident costs are used, the B/C ratio for Option 1 (barrier) is 2.6, Option 3 (extend) is 5.6, and Option 4 (modify opening) is 7.3.

Option 4 (modified opening) appears to be the best alternative. Constraints for this option include high potential for debris accumulation impeding water flow, soil erosion around the opening, and clear recovery area at the bottom of the slope. In selecting Option 3 (extend to clear zone), safety hazards should not be built into or around the new location (depressions, pockets, raised headwalls, humps, etc). Although Option 2 (shield with barrier) does not appear cost effective, barrier should be installed as a minimum if existing policies or practices dictate.

Example 49-10.2 Bridge Pier in Median.

Given: AADT = 30,000 with a 50% directional distribution
Growth = 4%
Design speed = 70 mph
4-lane divided highway/tangent section

Design options: Option 1 - no protection
Option 2 - W-beam guardrail with bullnose
Option 3 - concrete safety shape with tapered end section
Option 4 - concrete safety shape with sand barrels

Assumptions:

Use FHWA T 7570.1 accident cost
Project life = 20 years - 10 years for gravel barrels (Option 4)
Discount rate = 4%
No salvage value, except concrete safety shape (Option 4) where salvage value is approximately equal to new installation cost

Sensitivity Analysis:

1. See how changes to accommodate lane distribution affect the outcome.

   a. without lane distribution

   b. with lane distribution - run program separately for each lane (Figure 49-10E);
   c. use 30%-70% lane distribution; 4,500 (median lane) - 10,500 (right lane);

   d. with lane distribution - run program with user factor adjustment;

   e. use 0.62 (between 0.64 and 0.60 in Figure 49-10F).

Calculations:

Input Variable	Option 1	Option 2	Option 3	Option 4
Lateral distance (A)	35′	29′	34′	28′
Long. length (L)	50′	130′	210′	100′
Width (W)	3′	15′	5′	15′
Installation cost	$0	$10,000	$7,000	$17,000
Repair cost	$0	$100/acc	$0	$1000/acc
Maintenance cost	$0	$20/year	$10/year	$100/year
Salvage value	$0	$0	$0	$5,000
Severity index (face)	6.5	4.0	3.8	3.8
Severity index (side)	6.5	4.6	4.8	3.3

Summary:

Annualized cost without accommodating for lane distribution.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$24,486	$0	$24,486	n/a
Option 2	$12,154	$1,528	$13,682	8.1
Option 3	$10,938	$1,050	$11,988	12.9
Option 4	$5,154	$3,614	$8,768	5.4

Annualized cost with lane distribution - program run separately for each lane.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$15,946	$0	$15,946	n/a
Option 2	$8,012	$1,528	$9,540	5.2
Option 3	$7,152	$1,050	$8,202	8.4
Option 4	$3,426	$3,576	$7,002	3.5

Annualized cost with lane distribution - adjusting with user factor.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$15,180	$0	$15,180	n/a
Option 2	$7,536	$1,522	$9,058	5.0
Option 3	$6,780	$1,050	$7,830	8.0
Option 4	$3,174	$3,594	$6,768	3.3

Discussion:

Accident, agency and total equivalent uniform annual cost (EUAC) are shown for each option. The B/C ratios compared with no protection (Option 1) are also shown. The computer printout shows agency and accident cost for one direction. These costs are doubled assuming the other side of the piers are treated the same for both directions and the piers are in the center of the median.

Changing the analysis method to accommodate lane distribution lowers the B/C ratio for each option. The accident and agency costs are higher without lane distribution, because the model assigns 15,000 ADT to the lane closest to the obstacle (in this case the median lane). In adjusting for lane distribution, the EUAC are lower because most of the traffic will be in the right lane. This is an additional 12 feet further and therefore less probable of reaching the obstacle. EUAC and B/C ratios are slightly different between the user factor method and running the program separately for each lane. The analyst could easily check the sensitivity between methods by changing the user factor. The range would vary between running the model without lane distribution (user factor = 1.0) and with the lane distribution (user factor = value in Figures 49-10F and 49-10G).

All three improvements are cost effective compared with the no-protection alternative. Option 3 (concrete safety shape with tapered end section) has the highest B/C ratio. Option 4 (concrete safety shape with sand barrels) has the lowest EUAC. Each of these options may have other advantages and disadvantages which should be investigated before making the final decision.

Example 49-10.3 Ditch Along Roadside of 4-Lane Divided Highway

Determine the most cost-effective alternative.

Given: AADT = 13,000 with a 50% directional distribution
Growth = 2%
Design speed = 70 mph
4-lane divided highway/tangent section

Design options: Option 1- no protection
Option 2- W-beam guardrail
Option 3- install pipe and re-grade to 6:1/6:1 ditch section

Assumptions: Use FHWA T 7570.1 accident costs
Project life = 20 years
Discount rate = 4%
No salvage value
User factor 0.89 to accommodate lane distribution

Sensitivity Analysis:

1. Maintenance has pipe in stock and can do Option 3 with a 20% savings. See how a change in installation cost affects the outcome (Option 3a).

2. See how a change in accident cost affects the outcome (*RDG* default values - FHWA T 7570.1).

Input Variable	Option 1	Option 2	Option 3
Lateral Distance (A)	35′	29′	34′
Long. Length (L)	50′	130′	210′
Width (W)	3′	15′	5′
Installation Cost	$0	$10,000	$7,000
Repair Cost	$0	$100/acc	$0
Maintenance Cost	$0	$20/year	$10/year
Severity Index (Face)	6.5	4.3	4.3
Severity Index (Side)	6.5	4.8	4.8

Summary: Annualized cost - FHWA T 7570.1 accident costs.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$3,913	$0	$3,913	n/a
Option 2	$2,507	$759	$3,266	1.9
Option 3	$2,410	$525	$2,935	2.9
Option 4	$2,410	$422	$2,832	3.6

Annualized cost - *RDG* accident costs.

Alternatives	Accident Cost	Agency Cost	Total Cost	B/C Ratio
Option 1	$1,581	$0	$1,581	n/a
Option 2	$1,117	$759	$1,875	0.6
Option 3	$1,081	$525	$1,606	1.0
Option 4	$1,081	$422	$1,503	1.2

Discussion:

In changing from the FHWA T 7570.1 accident costs to the *RDG* accident costs, the following occurs: The decrease in the accident cost decreases the benefit-cost ratio by a factor of 3. The benefit (reduced accidents from existing condition - Option 1) decreases for each option because of the lower relative accident cost. In most cases, using a lower accident cost will not change the order of which option has the highest B/C ratio, but the B/C ratio may change significantly for an object with a high severity index. In this case, Option 3a has the highest B/C ratio with either set of accident costs.

If the equivalent uniform annualized cost (EUAC) method is used, Option 3a is still the best choice. In fact, using the *RDG* accident costs, Options 2 and 3 are both less desirable than Option 1. Only Option 3a has an EUAC less than Option 1.

As mentioned in the previous examples, each option may have other advantages and disadvantages that should be studied before making the final decision. Selection of the best option should be based on the results of the model, additional information and good engineering judgment.

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49-10.07 Application of ROADSIDE to Non-Level Roadsides (Slope Correction for Cost-Effectiveness Calculations)

Figure 49-2A provides the recommended clear zone ranges for various design speeds and for various side slope conditions. It also recommends different ranges for various traffic volumes, but this is a cost-effectiveness consideration rather than a safety need.

Using the information, a series of factors have been developed to input into the ROADSIDE computer program to better describe the effective lateral clearance (the "A" dimension).

It would then seem logical that, to achieve the same degree of safety and probability of accidents, the relationship between required clear zone distances could be used to develop factors to be multiplied to the actual lateral offset distance to derive the effective lateral clearance.

Assuming that the ROADSIDE program assumes a relatively flat side slope, the "flatter than 6:1" columns would have a correction factor of 1.0. The values in the other columns would become the denominators, and the values in the "flatter than 6:1" columns would become the numerators. The resulting fraction would be the factor to multiply the actual lateral clearance by to get the effective clearance.

Using the methodology described above, the factors become the following:

Clear Zone Adjustment Factors									
Design Speed	Cut Slopes					Fill Slopes			
	3:1	4:1	5:1	6:1	Flatter Than 6:1		6:1	5:1	4:1
≤ 40	1.0	1.0	1.0	1.0	1.0	1.0	0.93	0.93	0.81
45-50	1.23	1.14	1.0	1.0	1.0	1.0	0.89	0.80	0.62
55	1.33	1.25	1.11	1.0	1.0	1.0	0.91	0.83	0.67
60	1.63	1.44	1.18	1.08	1.0	1.0	0.87	0.81	0.65
65-70	1.56	1.27	1.17	1.08	1.0	1.0	0.88	0.82	0.67

When the ROADSIDE program asks for the lateral distance, A, one would multiply the plan or actual distance by the slope correction factor to get the effective lateral clearance. For example, a fixed object located 16 feet off the traveled way on a 5:1 fill slope on a highway with a design speed of 45 mph would be effectively 16 ft x 0.80 or 12.8 ft away. The 12.8 ft should be the value used for cost-effectiveness calculations.

49-11.0 ASSUMPTIONS FOR EMBANKMENT WARRANT FIGURES

The Figures 49-3B series provides warrants for guardrail on an embankment based on embankment height, slope, and design-year AADT. These figures were developed using the computer program ROADSIDE, as described in Section 49-10.0. This Section discusses the variables and assumptions that were used to develop the Figures 49-3B series. The line numbers listed below refer to the line numbers for imputing data into ROADSIDE; see Figure 49-10B. The following steps were used in the calculations.

1. Guardrail Calculations. ROADSIDE was first used to determine the present worth of providing guardrail along a 100-ft embankment. In addition to the following, Figures 49-11A through Figure 49-11F provide the assumptions used to develop these figures.

   a. Line 2. Figures 49-11A through 49-11F provide the design-year traffic volumes selected by the Department. The current traffic volumes were used in the program. A 2% traffic growth factor per year was assumed.

   b. Line 3. The calculations were run assuming a 2-lane, undivided facility with 12-ft width travel lanes.

   c. Line 4. The roadway was assumed to be on a tangent and in level terrain.

   d. Line 6. The English-units design speed was used.

   e. Line 7. The lateral location of the guardrail from the edge of the travel lane was assumed to be 10 ft for AADT between 700 and 1500, and 42 ft for AADT greater than 1500. The longitudinal length of the guardrail was calculated to be 1000 ft + $2*L_R$, where L_R is from Figure 49-5F. The width of guardrail was assumed to be 2 ft.

   f. Line 9. The severity indices from Figures 49-10H and 49-10 I for the guardrail face and the terminal ends were interpolated. The interpolations are shown in Figure 49-11G, Severity Indices. For an AADT less than 6000 and a design speed of 45 mph or lower, a buried-end terminal was used. For an AADT of 6000 or greater and a design speed of 50 mph or higher, a FHWA approved proprietary guardrail end treatment (CAT) was assumed. No corner impacts were assumed.

   g. Line 10. The project life for the guardrail installation was assumed to be 10 years with a 4% discount rate.

h. Line 11. The installation cost varies according to the design speed and AADT; see Figures 49-11A through 49-11F. Installation costs were taken from the *INDOT Catalog of Unit Price Averages for Roads - Bridges - Traffic*.

i. Line 12. The repairs costs in Figure 49-10Q were used.

j. Line 13. No maintenance costs were assumed.

k. Line 14. No salvage value was assumed.

2. <u>Embankment Calculations</u>. ROADSIDE was also used to determine an equivalent embankment severity index for an embankment without guardrail. The severity index for the embankment was selected to match the present worth of the guardrail using the assumptions in Figures 49-11A through 49-11F and the following:

   a. Line 2. Figures 49-11A through 49-11F provide the design-year traffic volumes selected by the Department. The current traffic volumes were used in the program. A 2% traffic growth factor per year was assumed.

   b. Line 3. The calculations were run assuming a 2-lane, undivided facility with 12-ft width travel lanes.

   c. Line 4. The roadway was assumed to be on a tangent and in level terrain.

   d. Line 6. The English-units design speed was used.

   e. Line 7. The lateral location of the embankment from the edge of the travel lane was assumed to be10 ft for AADT between 700 and 1500, and 12 ft for AADT greater than 1500. The embankment was assumed to be 1000 ft long. For calculation purposes, the width of the embankment was assumed to be 25 ft.

   f. Line 9. For an embankment, the severity index was selected to match the present worth for the guardrail installation.

   g. Line 10. The project life for the embankment was assumed to be 20 years with a 4% discount rate.

   h. Line 11. No installation costs were assumed because the embankment would also be in place for guardrail installations.

   i. Line 12. No repairs costs were assumed.

j. Line 13. No maintenance costs were assumed.

k. Line 14. No salvage value was assumed.

3. Slope Equivalents. Using Figure 49-10K and interpolating for the metric-units design speed, the slope indices were developed and are provided in Figure 49-11G, Severity Indices. The higher-range indices were assumed to be for an embankment height of at least 5.0 m. The midrange indices were assumed to be for a height of 6.5 ft. The lower-range indices were assumed to be for an embankment height of 1.5 ft. Using Figure 49-11G and the equivalent embankment severity index shown in Figures 49-11A through 49-11F, the equivalent slope can be determined for each embankment height and AADT.

3. Data Plotting. The data points determined in Step 3 were used to develop the Figures 49-3B series. AASHTO *Roadside Design Guide* Figure 5.1 was also imposed on the charts as a lower boundary for where guardrail is required. The AADT of 18,000 was assumed to be the maximum traffic volume that can be reasonably obtained on a 2-lane facility and, therefore, is considered to be a lower boundary.