NOTE: Users should consult Design Memoranda listed on the title sheet of each affected chapter for details related to revisions.
Preface

Part 3, Roadway Design, of the Indiana Design Manual has been developed to provide uniform design practices for Department and consultant personnel preparing contract plans for Department projects. The roadway designer should attempt to meet all criteria presented in the Manual. However, the Manual should not be considered a standard which must be met regardless of impacts.

Part 3 of the Manual presents most of the information normally required in the design of a roadway project; however, it is impossible to address every situation which may be encountered. Therefore, designers must exercise good judgment on individual projects and, frequently, they must be innovative in their approach to roadway design. This may require, for example, additional research in the highway literature.
## CHAPTER 302

### Roadway Design Guidelines

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NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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CHAPTER 40

BASIC DESIGN CONTROLS

40-1.0 HIGHWAY SYSTEMS

40-1.01 Functional-Classification System

The functional classification concept is a determining factor in highway design. In this concept, each highway is grouped by the character of service it provides. Functional classification recognizes that the public-highway network serves two basic and often conflicting functions, access to property and travel mobility. Each highway or street will provide varying levels of access and mobility, depending upon its intended service. In the functional-classification scheme, the overall objective is that the highway system, if viewed in its entirety, will yield an optimum balance between its access and mobility purposes. If this objective is achieved, the benefits to the traveling public will be maximized.

The functional-classification system provides the framework for determining the geometric design of an individual highway or street. Once the function of the highway facility is defined, the designer can select an appropriate design speed, roadway width, roadside-safety elements, amenities, and other design values. Part V is based upon this systematic concept of determining highway design.

The Planning Division’s Office of Highway Statistics has functionally classified each public highway and street. To design a project, it is necessary to determine the predicted functional class of the highway or street for the selected design year (e.g., 20 years beyond the project completion date).

40-1.01(01) Arterial

An arterial highway is characterized by a capacity to quickly move relatively a large volume of traffic and an often restricted function to serve abutting properties. The arterial system provides for high travel speed and the longest trip movements. A rural arterial provides connections between major urban areas and provides a level of service suitable for statewide or interstate travel. The rural-arterial system provides integrated, continuous movements without the need for stub connections.
In an urban area, the arterial system serves the major centers of activity within the urban area, carries the highest traffic volume and longest trip movements, and serves both major intra-city and through trips. The rural and urban arterial systems are connected to provide continuous through movements at approximately the same level of service.

The arterial functional classification is subdivided into principal and minor categories for rural and urban areas, as follows.

1. **Principal Arterial.** In a rural or urban area, the principal arterial provides the highest traffic volume and the greatest trip lengths. A principal arterial can be further subdivided as follows.

   a. Freeway. The freeway, which includes each Interstate highway, is the highest level of arterial. This type of facility is characterized by full control of access, high design speeds, and a high level of driver comfort and safety. For these reasons, a freeway is considered a special type of highway within the functional-classification system, and separate design criteria have been developed for it.

   b. Other Principal Arterial or Expressway. This type of facility may be 2 or 4 lanes, with or without a median. Partial control of access is desirable along this type of facility and, if a divided highway, this is termed an expressway. The level of geometric design is often equivalent to that of a freeway (e.g., 12-ft lane widths are required).

2. **Minor Arterial.** In a rural area, a minor arterial will provide a mix of interstate and interregional travel service. In an urban area, a minor arterial may carry local bus route and provide intra-community connections, but it will not, for example, penetrate a neighborhood. If compared to the principal arterial, the minor arterial provides lower travel speed, accommodates shorter trip lengths and lower traffic volume, but it provides more access to property.

**40-1.01(02) Collector**

A collector route is characterized by a roughly even distribution of its access and mobility functions. Traffic volume and speed will be somewhat lower than that for an arterial. In a rural area, a collector serves intra-regional needs and provides connections to the arterial system. All cities and towns within a region will be connected. In an urban area, a collector acts as an intermediate link between the arterial system and points of origin and destination. An urban
collector penetrates a residential neighborhood or commercial or industrial area. Local bus routes often include collector streets.

**40-1.01(03) Local Road or Street**

Each public road or street not classified as an arterial or collector is classified as a local road or street. A local road or street is characterized by its many points of direct access to adjacent properties and its relatively minor value in accommodating mobility. Speed and traffic volume are low and trip distances are short. Through traffic is often deliberately discouraged.

**40-1.01(04) Recreational Road**

A recreational road, which is a subset of the local-road system, provides access to a campground, park, boat-launching ramp, picnic area, or scenic or historic site. It is designed to protect and enhance the existing aesthetic, ecological, environmental, or cultural amenities that form the basis for distinguishing each recreational site or area. Because of its unique functional purpose, specific geometric design criteria have been developed for a recreational road. These are provided in Chapter 51.

**40-1.02 Urban Design Subcategories by Type of Area**

The functional-classification system is divided into urban and rural categories. However, an urban or rural designation may not be sufficiently specific to determine the appropriate project design, especially in an urban area. Therefore, the design criteria described for an urban project in Chapters 53 through 56 are further divided by the type of area where the project is located. This refinement to the highway-design process will allow the designer to tailor an urban project to the constraints of the surrounding environment.

Within an urbanized or urban area, the selection of design values will depend upon the design subcategory of the facility. A separate design is appropriate for a suburban, intermediate, or built-up classification. The following provides a description of the subcategories.

1. **Suburban.** This area is located at the fringe of an urbanized or small urban area. The predominant character of the surrounding environment is residential, but it may also include a considerable number of commercial establishments, especially strip development along a suburban arterial. There may also be at least one industrial park. The motorist has a significant degree of freedom but, nonetheless, must also devote attention to entering and exiting vehicles. Roadside development is characterized by low
2. Intermediate. As its name implies, an intermediate area is classified between a suburban and a built-up area. The surrounding environment may be residential, commercial, or industrial, or a combination of these. The extent of roadside development will have a significant impact on the selected speeds of motorists. The increasing frequency of intersections is also a control on average travel speeds. Pedestrian activity has become a significant design consideration, and sidewalks and crosswalks at intersections are common. The available right of way will restrict the practical extent of roadway improvements.

A local or collector street in an intermediate area has a posted speed limit ranging from 30 to 45 mph. The frequency of signalized intersections has increased substantially if compared to a suburban area. An arterial in an intermediate area will often have intensive commercial development along its roadside. The posted speed limit ranges from 35 to 50 mph. An arterial has a number of signalized intersections per mile.

3. Built-up. This refers to the central business district within an urbanized or small urban area. The roadside development has a high density and is often commercial. However, a substantial number of roads and streets in a built-up area pass through a high-density, residential environment (e.g., apartment complexes, row houses). Access to property is the primary function of the road network. Pedestrian considerations may be as important as vehicular considerations, especially at intersections. Right of way for roadway improvements is not available.

Because of the high density of development in a built-up area, the distinction between the functional classifications becomes less important in considering signalization and speeds. The primary distinction among the three functional classifications is often the relative traffic volume and, therefore, the number of lanes. As many as half the intersections may be signalized. The posted speed limit ranges from 25 to 35 mph.

See Section 40-1.01 for definitions of the functional classifications.
40-1.03 Federal-Aid System

The Federal-aid system previously consisted of those routes which were eligible for categorical Federal highway funds. The Department, working with local governments and in cooperation with FHWA, designated the eligible routes. The criteria were based on the relative importance of the highway route and the anticipated functional classification 5 to 10 years in the future. United States Code, Title 23, described the applicable Federal criteria for establishing the Federal-aid system.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 implemented a major realignment of the Federal-aid system. The system had been divided into Interstate, primary, secondary, and urban Federal-aid systems. Separate categories of Federal funds were available for eligible Federal-aid projects on each system. The following describes the Federal-aid system created by ISTEA.

40-1.03(01) National Highway System

The National Highway System (NHS) is a system of highways determined to have the greatest national importance to transportation, commerce, and defense in the United States. It consists of the Interstate highway system, logical additions to the Interstate system, selected other principal arterials, and other facilities which satisfy the requirements of one of the subsystems within the NHS. The National Highway System has been revised recently to include designated Intermodal Connectors which serve major ports, airports, public transportation and transit facilities, interstate bus terminals and rail and intermodal transportation facilities. Two Strategic Highway Network (STRAHNET) facilities have also been designated. The NHS represents approximately 4 to 5% of the total public-road mileage in the United States. Specifically, the NHS includes the following subsystems. A specific highway route may be on more than one subsystem.

1. **Interstate System.** The current Interstate highway system retains its separate identity within the NHS. There are provisions to add mileage to the existing Interstate subsystem.

2. **Other Principal Arterials.** These are highways in rural and urban areas which provide access between an arterial and a major port, airport, public transportation facility, or other intermodal transportation facility.
3. **Strategic Highway Network.** This is a network of highways which is important to the United States’ strategic defense policy and which provide defense access, continuity, and emergency capabilities for defense purposes.

4. **Major Strategic Highway Network Connectors.** These are highways which provide access between major military installations and highways which are part of the Strategic Highway Network.

5. **Intermodal Connectors.** These are highways connecting NHS routes to major ports, airports, public transportation and transit facilities, interstate bus terminals, and rail or other intermodal transportation facilities.

Maps illustrating these routes and their locations are available and accessible on the Department’s website at [www.in.gov/indot/2350.htm](http://www.in.gov/indot/2350.htm).

The 1991 ISTEA mandated that the Department, in cooperation with other jurisdictional agencies, develop and implement transportation management systems. These include management systems for pavements, bridges, traffic congestion, highway safety, public transportation facilities and equipment, and intermodal transportation facilities or systems. Chapter 4 discusses INDOT’s development of transportation management systems.

Local and military authorities should be consulted to verify the project traffic assignments (AADT), truck volume (% AADT commercial), and types of trucks that use the facility to ensure that the proper design vehicles are used for geometric design of the project.

**40-1.03(02) Surface Transportation Program**

The Surface Transportation Program (STP) is a block-grant type program that may be used by the State or a local agency for a road including an NHS facility that is not functionally classified as a local or rural minor collector. Such a road is referred to as a Federal-aid route. A bridge project funded through this program is not restricted to a Federal-aid route. A transit capital project is also eligible for Federal aid through the STP program.
40-1.03(03) Bridge Replacement and Rehabilitation Program

The Bridge Replacement and Rehabilitation Program (BRRP) has retained its separate identity within the Federal-aid program. BRRP funds are eligible for work on a bridge regardless of the road’s functional classification.

40-1.04 Jurisdictional System

The State includes approximately 92,000 mi of public roads. The network has been divided into jurisdictional systems based on the organization or agency responsible for highway or street improvements and for maintenance.

40-1.04(01) State Highway System

The State highway system consists of all highways under the jurisdiction of the Indiana Department of Transportation. This system includes all Interstate highways, the Indiana Toll Road, the majority of the facilities on the National Highway System, and other State and U.S. routes not on the NHS. The State highway system consists of about 12%, or 11,350 mi, of all public roads and streets. These routes are the most important highways in the State, have the greatest traffic volume, and operate at the highest speeds.

40-1.04(02) County Road System

Each county government is responsible for all roads within its boundaries which are not on the State highway system, but is not responsible for the streets within incorporated cities or towns within the county. There are 66,078 mi of county-maintained roads in the State. In addition to the county-road system, each county is responsible for maintenance and improvements of bridges on city or town roads or streets. INDOT is responsible for administering Federal funds which are available for highway improvements on eligible county routes. The construction of a county-road bridge over a State or Interstate route is the responsibility of INDOT. The maintenance of such a bridge is the responsibility of INDOT. The maintenance of a bridge which carries a railroad over a road or street is the responsibility of the railroad company.
40-1.04(03) City or Town Street System

The city or town street system consists of all public streets within corporate limits except those on the State highway system. There are 14,519 mi of city- and town-maintained streets in the State. The extensions of these routes outside the corporate limits, but still within an urbanized or small urban area, are the responsibility of the county. INDOT is responsible for administering Federal funds which are available for highway improvements on eligible city or town streets.

40-1.04(04) DNR Recreational Roads

The Indiana Department of Natural Resources (IDNR) is responsible for maintaining roads within State public recreational areas. INDOT may be responsible for the design and construction of these facilities in cooperation with IDNR.

40-1.05 National Truck Network

The Surface Transportation Assistance Act (STAA) of 1982 required that the U.S. Secretary of Transportation, in cooperation with the Department, designate a national network of highways which allow the passage of trucks of specified minimum dimensions and weight. The objective of the STAA is to promote uniformity throughout the nation for legal truck sizes and weights on a National Truck Network. The Truck Network includes all Interstate highways and significant portions of the former Federal-aid primary system built to accommodate large-truck travel. The STAA requires that reasonable access be provided along other designated routes to the STAA commercial vehicles from the National Truck Network to terminals and to facilities for food, fuel, repair, and rest, and, for household-goods carriers, to points of loading and unloading.

Under State statute, each principal arterial is available to commercial vehicles with the dimensions authorized by the STAA, subject to local restrictions. The State has enacted legislation that stipulates that each public road is legally available to STAA vehicles, subject to local restrictions. The geometric-design criteria provided in the applicable Part V Chapters reflect the impact of the STAA vehicles on road design. For example, a 12-ft lane width is required for each highway on the National Truck Network.

Figure 40-1B provides the National Truck Network in Indiana. The National Truck Network is also available as a separate layer on the INDOT Roadway Inventory map at http://gis.in.gov/apps/DOT/RoadwayInventory/.
40-2.0 TRAFFIC-VOLUME CONTROLS

40-2.01 Definitions

1. **Average Annual Daily Traffic (AADT).** The total yearly volume in both directions of travel divided by the number of days in a year.

2. **Average Daily Traffic (ADT).** The calculation of average traffic volume in both directions of travel in a time period longer than one day and shorter than one year and divided by the number of days in that time period. Although incorrect, ADT is often used interchangeably with AADT.

3. **Capacity.** The maximum number of vehicles which can reasonably be expected to traverse a point or uniform section of a road during a given time period under prevailing roadway, traffic, and control conditions. The time period used for analysis is 15 min. Capacity corresponds to Level of Service E.

4. **Delay.** The primary performance measure on an interrupted-flow facility, especially at a signalized intersection. For this element, average stopped-time delay is measured, which is expressed in seconds per vehicle.

5. **Density.** The number of vehicles occupying a given length of lane, averaged over time. It is expressed as vehicles per mile per lane.

6. **Design Hourly Volume (DHV).** The 1-h volume in both directions of travel in the design year selected for determining the highway design. The DHV is the 30th highest hourly volume during the design year. For capacity analysis, the DHV is converted to an hourly flow rate based on the maximum 15-min flow rate during the design hour.

7. **Service Flow Rate.** The maximum hourly vehicular volume which can pass through a highway element at the selected level of service.

8. **Directional Design Hourly Volume (DDHV).** The 1-h volume in one direction of travel during the design hour in the selected design year.

9. **Directional Distribution (D).** The division, by percent, of the traffic in each direction of travel during the design hour or day.

10. **Level of Service (LOS).** A qualitative concept which has been developed to characterize an acceptable degree of congestion as perceived by the motorist. In the *Highway}
Capacity Manual, the qualitative descriptions of each level of service (A to F) have been converted into quantitative measures for the capacity analysis for each highway element as follows:

a. freeway mainline;
b. freeway mainline ramp junction;
c. freeway weaving area;
d. interchange ramp;
e. 2-lane, 2-way rural highway;
f. rural highway of 4 or more lanes;
g. signalized intersection;
h. unsignalized intersection; and
i. urban or suburban arterial.

Chapters 53 through 56 provide guidelines for selecting the level of service for capacity analysis in road design.

11. Peak-Hour Factor (PHF). A ratio of the volume occurring during the peak hour to the maximum rate of flow during a given time period within the peak hour, typically, 15 min.

12. Percent Trucks (T). A factor which reflects the percentage of heavy vehicles (trucks, buses, and recreational vehicles) in the traffic stream during the design hour or day. For geometric design and capacity analysis, a truck is defined as a vehicle with six or more tires. Data on trucks are compiled and reported by the Planning Division’s Traffic Monitoring Team.

13. Rate of Flow. The equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than one hour, typically, 15 min.

40-2.02 Design Year Selection

40-2.02(01) Roadway Design

A highway should be designed to accommodate the traffic volume expected to occur within the life of the facility under reasonable maintenance. This involves projecting the traffic conditions for a selected future year. The recommended design year is provided in Figure 40-2A. The design year is measured from the expected letting date for construction. Future traffic volume for each State highway is provided by the Planning Division’s Traffic Monitoring Team.
40-2.02(02) Other Highway Elements

The following provides the recommended criteria for consideration of a design year for highway elements other than road design:

1. **Bridge or Underpass.** The structural life of a bridge may be 50 years or more. For a new bridge, including a bridge replacement, the initial clear-roadway width of the bridge or underpass will be based on the 20-year traffic-volume projection beyond the original projected letting date for construction.

   A bridge-rehabilitation project is that for which a significant amount of the existing substructure or superstructure will remain in place. For a bridge-rehabilitation project which includes significant improvements to all or part of the superstructure including full bridge-deck replacement, the clear-roadway width will be based on the 20-year traffic-volume projection beyond the original projected letting date for construction. A project which includes a bridge-deck overlay may be based on the 10-year projected traffic volume. For a bridge-rehabilitation project which includes only improvements to the substructure, the bridge will be evaluated as an existing bridge to remain in place. See Chapters 53 through 56 for specific criteria.

2. **Right-of-Way Grading.** The designer should consider potential right-of-way needs for the anticipated long-term corridor growth for a year considerably beyond that used for roadway design. No specific design year is recommended. However, in selecting an initial median width on a divided highway, for example, the designer should evaluate the potential need for future expansion of the facility to add through travel lanes. Other examples include potential future interchanges and potential conversion of a 2-lane, 2-way facility to a divided highway of 4 or more lanes.

3. **Drainage Design.** Drainage appurtenances are designed to accommodate a flow rate based on a specific exceedance probability (EP) or frequency of occurrence. The selected EP or frequency will be based on the functional classification of the facility and the specific drainage appurtenance (e.g., culvert). Chapter 203 provides the Department’s criteria for selecting an EP for drainage.

4. **Pavement Design.** The pavement structure is designed to withstand the vehicular loads it will sustain during the design analysis period without appearing below selected terminal pavement serviceability. Chapter 304 provides the Department’s criteria for selecting a design year for pavement design.
40-2.03 Design-Hourly-Volume Selection

For most geometric design elements which are impacted by traffic volume, the peaking characteristics are most significant. The highway facility should be able to accommodate the design hourly volume (adjusted for the peak-hour factor) at the selected level of service. This design hourly volume (DHV) will affect many design elements including the number of travel lanes, lane and shoulder widths, and intersection geometrics. The designer should also analyze the proposed design using morning and evening DHVs separately. This can have an impact on the geometric design of the highway.

The 30th highest hourly volume in the selected design year will be used to determine the DHV for design purposes.

40-2.04 Capacity Analysis

40-2.04(01) Objective

The highway mainline, intersection, or interchange should be designed to accommodate the selected design hourly volume (DHV) at the selected level of service (LOS). This may involve adjusting highway factors which affect capacity until a design is found that will accommodate the DHV. The detailed calculations, factors, and methodologies are provided in the *Highway Capacity Manual (HCM)*. Chapter 41 provides additional information which the Department has adopted for the use of the HCM. The service flow rate of the facility is calculated. Capacity assumes a LOS of E. Service flow rate is the maximum volume of traffic that a proposed highway of given geometrics is able to serve without the degree of congestion appearing below a preselected LOS. This is always higher than a LOS of E.

The HCM has established measures of effectiveness for the level-of-service definition for each highway element for each type of highway facility. These are provided in Figure 40-2B. For each measure, the HCM will provide the analytical tools required to calculate the numerical value.

The following provides the simplified procedure for conducting a capacity analysis for the highway mainline.

1. Select the design year (Section 40-2.02).
2. Determine the DHV (Section 40-2.03).

3. Select the Level of Service (Chapters 53 through 56).

4. Document the proposed highway geometric design (lane width, clearance to obstructions, length of weaving section, number and width of approach lanes at an intersection, etc.).

5. Using the HCM, analyze the capacity of the highway element for the proposed design as follows:
   
a. determine the maximum flow rate under ideal conditions;
   
b. adjust the maximum flow rate for prevailing roadway, traffic, and control conditions; and
   
c. calculate the service flow rate for the selected level of service.

6. Compare the calculated service flow rate to the DHV. If the DHV is less than or equal to the service flow rate, the proposed design will satisfy the objectives of the capacity analysis. If the DHV exceeds the service flow rate, the proposed design will be inadequate. The designer should either adjust the highway design or should adjust one of the capacity elements (e.g., the selected design year or the level-of-service goal).

40-2.04(02) Responsibility

For a State highway project, the Office of Environmental Services or its consultant is responsible for performing the required capacity analysis.

For a consultant-designed project on a non-State highway, the capacity analysis may be performed by either the local jurisdiction or the consultant.

40-3.0 SPEED

40-3.01 Definitions

1. Design Speed. Design speed is the maximum safe speed that can be maintained over a specified section of highway if conditions are so favorable that the design features of the highway govern. A design speed is selected for each project which will establish criteria
for design elements including horizontal and vertical curvature, superelevation, and sight distance. Section 40-3.02 discusses the selection of design speed. Chapter 53 provides specific design-speed criteria for a new construction or reconstruction project. Chapters 54 through 56 provide the design-speed criteria for a project on an existing highway.

2. **Low Speed.** For geometric design purposes, low speed is defined as 45 mph or lower.

3. **High Speed.** For geometric design purposes, high speed is defined as 50 mph or higher.

4. **Average Running Speed.** Running speed is the average speed of a vehicle over a specified section of highway. It is equal to the distance traveled divided by the running time (the time the vehicle is in motion). The average running speed is the distance summation for all vehicles divided by the running time summation for all vehicles.

5. **Average Travel Speed.** Average travel speed is the distance summation for all vehicles divided by the total time summation for all vehicles. Average running speed includes only the time the vehicle is in motion. Therefore, on an uninterrupted-flow facility which is not congested, average running speed and average travel speed are equal.

6. **Operating Speed.** Operating speed, as defined by AASHTO, is the highest overall speed at which a motorist can safely travel a given highway under favorable weather conditions and prevailing traffic conditions while at no time exceeding the design speed. Therefore, for low-volume conditions, operating speed equals design speed. This term has little or no usage in geometric design.

7. **85th-Percentile Speed.** The 85th-percentile speed is the speed below which 85 percent of vehicles travel on a given highway. The most common application of the value is its use as one of the factors for determining the posted, regulatory speed limit of a highway section. Field measurements for the 85th-percentile speed will be conducted during off-peak hours when motorists are free to select their desired speed.

8. **Posted Speed Limit.** If needed, the INDOT district Office of Traffic conducts the traffic engineering studies on the State highway system to select a posted speed limit. If a study is performed, on either the State or local system, the posted speed limit is based on the following:
   
   a. the 85th-percentile speed;

   b. the design speed used during project design;
c. road-surface characteristics, shoulder condition, grade, alignment, and sight distance;

d. functional classification and type of area;

e. type and density of roadside development;

f. accident experience during the previous 12 months; and

g. parking practices and pedestrian activity.

On a new-construction or reconstruction project, the posted speed limit will be equal to the design speed used in design, if this does not exceed the legal speed limit. A traffic engineering study may be conducted to assist in the determination of the posted speed limit. This procedure applies to either a State or non-State facility.

9. **Legal Speed Limit**. The legal speed limit is that set by the Indiana Statutes which applies to a public road which does not have a posted speed limit. Section 40-3.02 provides legal speed limits adopted by the State of Indiana. An advisory speed sign is not a regulatory sign. Hence, it is meaningless for determining the posted speed limit.

### 40-3.02 Design-Speed Selection

#### 40-3.02(01) Geometric Design Considerations

From a geometric design perspective, the selected design speed is based on the following design elements.

1. **Functional Classification**. A facility regarded as more important is designed with a higher design speed than a facility regarded as less important.

2. **Urban or Rural**. The design speed in a rural area is higher than that in an urban area. This is consistent with the likelihood of fewer constraints occurring in a rural area (e.g., less development).

3. **Terrain**. The flatter the terrain, the higher the selected design speed will be. This is consistent with the expected higher construction cost as the terrain becomes more rugged.
4. **Traffic Volume.** Design speed can vary by traffic volume. As traffic volume increases, a higher design speed is used. For example, the design speed on a rural collector varies according to traffic volume.

5. **Project Scope of Work.** A higher design speed is more applicable to a new-construction or reconstruction project than to a 3R project.

For geometric design application, the relationship between these road-design elements and the selected design speed reflects cost-effective considerations. For example, the higher the traffic volume, the more benefit to the traveling public from a higher design speed.

### 40-3.02(02) Regulatory Speed vs. Design Speed

Each public road is controlled by a regulatory speed limit, either through posted speed-limit signs or with a legal speed limit established in the State statutes; see Section 40-3.02(03). The following summarizes the relationship between the project design speed and the regulatory speed limit.

1. **General.** The design speed should equal or exceed the anticipated posted speed limit after construction, or the State legal speed limit for a non-posted highway.

2. **Non-Posted Rural Facility.** The maximum legal speed limit is 55 mph. A project on such a facility must be designed for 55 mph, or a traffic engineering study must be conducted to determine if a lower design speed is appropriate. If the project is designed for a lower speed than 55 mph, the road must be posted at the selected design speed between logical termini.

3. **Non-Posted Urban Facility.** The maximum legal speed limits, and corresponding minimum design speeds, are as follows:
   
   a. on a State highway, maximum legal speed limit 30 mph, minimum design speed 30 mph; and
   
   b. on a non-State highway, maximum legal daytime speed limit 55 mph, maximum legal nighttime speed limit 50 mph, minimum design speed 55 mph.

   As in a rural area, the minimum design speed must satisfy these criteria, unless a traffic engineering study indicates otherwise.
To avoid a potential conflict, the Office of Environmental Services should, early in project
development, coordinate the design-speed selection with the district Office of Traffic to assist in
establishing the anticipated posted speed limit of the completed facility. If the proposed design
speed from the Geometric Design Tables is less than the established posted speed limit, one of
the determinations must be made as follows:

1. increase the design speed to equal or exceed the established or anticipated posted speed
   limit; or

2. seek a design exception for the individual geometric design element (e.g., a horizontal
curve) which does not satisfy the established-speed-limit requirement.

**40-3.02(03) Legal Speed Limit**

The legal speed limits established by the State statutes are summarized below. Figure 40-3A
provides the legal speed limits for a non-Interstate facility.

1. **Maximum Speed Limit.** IC 9-21-5-2 and IC 9-21-5-6 set maximum speed limits which
   apply to vehicular speeds for all public roads. These maximum limits do not establish
   upper limits for geometric design speeds. The speed limits are as follows:

   a. 70 mph on an Interstate route outside an urbanized area;

   b. 65 mph on an Interstate route outside an urbanized area for a vehicle other than a
      bus having a gross weight greater than 26,000 lb;

   c. 60 mph on a rural facility of 4 or more lanes;

   d. 55 mph on a rural facility of 2 lanes;

   e. 55 mph on an Interstate route inside an urbanized area;

   f. 30 mph on a State highway in an urban area *;

   g. 30 mph on a non-State highway in an urban area, with maximums of 55 mph
daytime, and 50 mph nighttime *; and

   d. 15 mph in an alley, with a maximum of 30 mph *. 
* Requires an engineering and traffic investigation study to establish a maximum speed limit that is different from the value shown.

2. **Minimum Speed Limit (Non-State Facility).** IC 9-21-5-6 of the Statutes sets minimum speed limits which apply to a non-State facility which is not posted with a regulatory speed-limit sign. The speed limits are as follows.

   a. **Rural Area.** 30 mph, except as shown in Item 2.c. below **.

   b. **Urban Area.** 20 mph, except as shown in Items 2.c. and 2.d. below **.

   c. **School Zone.** A local authority may establish a speed limit within a school zone if the following conditions are satisfied.

      (1) The limit is not lower than 20 mph.

      (2) The limit is imposed only in the immediate vicinity of the school.

      (3) Children are present.

      (4) The speed zone is properly signed.

      (5) The Department has been notified by certified mail of the limit imposed.

   d. **Park or Playground.** A local authority may decrease the speed limit on an urban street to not lower than 15 mph, if the following conditions exist.

      (1) The street is located within a park or playground established under IC 36-10.

      (2) The boards established under IC 36-10-3 or IC 36-10-4, or the park authority established under IC 36-10-5 requests the local authority to decrease the limit.

      (3) The speed zone is properly signed.

   e. **Alley.** 5 mph **.
** Requires an engineering and traffic investigation study to establish a speed limit that is below the maximum. However, the lower limit can not be lower than the minimum value shown.

40-4.0 VEHICULAR CHARACTERISTICS

The physical and operational characteristics of vehicles using the highway are important controls in geometric design. These will vary according to the type of vehicle being considered. If a highway facility or intersection is being designed, the largest design vehicle likely to use that facility with considerable frequency should be used to determine the selected design values. See Chapter 46 for design-vehicle selection at an intersection.

Figure 40-4A provides information on dimensions for the standard design vehicles. Figures 40-4B and 40-4C provide illustrations for two combination trucks for application of the basic dimensions.

40-5.0 ACCESS-CONTROL DEFINITIONS

Access control is defined as the condition where a public authority fully or partially controls the right of abutting owners to have access to and from the public highway. The functional classification of a highway is partially determined by the degree of access it allows. Access control may be exercised by statute, zoning, right of way purchases, drive controls and permits, turning and parking regulations, or geometric design (e.g., grade separation or frontage road).

The definitions of the types of access control are as follows.

1. Full Control. Full control of access is achieved by giving priority to through traffic by providing access only at interchanges with selected public roads. At-grade crossings or drive connections are not allowed. This type of facility is termed a freeway. Full control of access maximizes the capacity, safety, and vehicular speeds on the freeway.

2. Partial Control. Partial control of access is an intermediate level between full control and no control. Priority is given to through traffic, but some at-grade intersections and drive connections are allowed. The proper selection and spacing of at-grade intersections and service connections will provide a balance between the mobility, safety, and access service of the highway. This type of facility is termed an expressway.
3. **No Control.** The use of the term *no* is actually a misnomer. Each highway warrants some degree of access control by permit or by design. If access points to other public roads and drives are properly spaced and designed, the adverse effects on highway capacity and safety will be minimized. These points should be located where they can best suit the traffic and land-use characteristics of the highway being designed. Their design should enable a vehicle to enter and exit safely with a minimum of interference to through traffic. Access control is exercised by the Department on a State highway or by a local jurisdiction on a non-State facility to determine where private interests may have access to and from the public-road system.

The designer should reference the following for more information on access-control regulations and design guidelines.

1. INDOT *Driveway Permit Handbook*.

2. Indiana Local Technical Assistance Program (LTAP) *Access Control for Local Roads and Streets in Small Cities and Rural Areas*.


4. INDOT *Standard Drawings*.

5. *Indiana Design Manual* Sections 46-8.0, 48-6.06, 48-1.03, and Chapter 86.

### 40-6.0 PROJECT SCOPE OF WORK

The project scope of work will reflect the basic intent of the highway project and will determine the overall level of highway improvement. This decision will determine which criteria in this Part will apply to the geometric design of the project.

#### 40-6.01 Definitions

**40-6.01(01) New Construction**

New construction is defined as horizontal and vertical alignment in a new location. An intersection or interchange which appears within the project limits of a new highway mainline or is relocated to a new point of intersection is considered new construction. Chapters 41 through 53 provide the Department’s criteria for new construction.
40-6.01(02) Complete Reconstruction, Freeway

Complete reconstruction of an existing freeway is defined as replacement of the existing facility. Complete reconstruction results in significant improvements to the freeway’s level of service, operational efficiency, and safety. Because of the significant level of work, Chapters 41 through 53 will apply to the design of a complete reconstruction project.

40-6.01(03) Partial Reconstruction (4R), Freeway

Partial reconstruction (4R) of an existing freeway is defined as work which includes one or more of the following improvements.

1. Over 30% of the travelway pavement area must be removed and replaced. Pavement rubblization with an overlay is considered to be one form of pavement removal and replacement.

2. A concrete overlay of at least 6 in., or an asphalt overlay of 8 in. or greater as measured at the point of greatest thickness over the existing travelway, is required.

3. The facility cannot adequately accommodate the current or projected (10-year) traffic demand and additional lanes are necessary.

4. Major revisions are necessary to the existing horizontal and vertical alignment requiring that over 30% of the travelway pavement must be replaced.

5. Bridge replacement or total bridge-deck replacement is required.

6. Bridge-deck widening is necessary due to added travel lanes on the approach.

7. Interchange upgrading is required to satisfy current and projected (20-year) traffic demands.

A partial 4R freeway project is to be designed in accordance with Chapter 54.
40-6.01(04) Reconstruction (4R), Non-Freeway

Reconstruction of an existing highway mainline includes the addition of travel lanes or major revisions to the existing horizontal and vertical alignment or reconstruction of a significant portion of the existing pavement structure. However, the highway will remain essentially within the existing corridor. The project may require right-of-way acquisition. A 4R project is undertaken because one or more of the following conditions exist along the highway.

1. Over 30% of the pavement area in the traveled way must be removed and replaced.

2. A concrete overlay of at least 6 in., or an asphalt overlay of 8 in. or greater as measured at the point of greatest thickness over the existing travelway, is required.

3. The facility cannot adequately accommodate its current or projected (10-year) traffic demand and additional lanes are necessary.

4. Major revisions are necessary to the existing horizontal and vertical alignment requiring more than 30% of the traveled way to be replaced.

5. Bridge replacement or total bridge-deck replacement is required.

6. Bridge-deck widening is necessary due to added travel lanes on the approach.

7. Major interchange upgrading is necessary to satisfy current and projected (20-year) traffic demands at an acceptable level of service. However, an analysis may determine that interim improvements are cost effective.

8. Work planned on adjoining sections of the highway involves reconstruction for an appreciable length of the highway requiring reconstruction to achieve roadway-design consistency along the route between logical termini.

The final decision on selecting a 4R scope of work will be made based on the Department’s long-range plans for upgrading the State’s highway system. See Section 40-6.02 for more information.

Because of the significant level of work for reconstruction, the design will be based on the criteria for new construction. Therefore, Chapters 41 through 53 will apply to a reconstruction (4R) project.

An added-travel-lanes project should be classified as a 4R project.
40-6.01(05) 3R Project, Freeways

A 3R project (resurfacing, restoration, rehabilitation) on an existing freeway is intended to extend the service life of the existing facility and to enhance highway safety. A 3R project should make cost-effective improvements to the existing geometrics where practical. Right of way acquisition is rarely necessary. Improvements include the following:

1. pavement resurfacing;

2. full-depth pavement reconstruction, if the reconstructed pavement area is 30% or less of the traveled way;

3. widening existing travel lanes or shoulders;

4. upgrading the structural strength of shoulders;

5. improving the superelevation of existing horizontal curves;

6. adding an auxiliary lane;

7. improving roadway delineation;

8. upgrading roadside safety;

9. increasing the length of acceleration and deceleration lanes at an interchange;

10. widening an existing bridge as part of a bridge-reconstruction project;

11. upgrading or replacing bridge railing;

12. overlaying a bridge deck;

13. preservation of bridge substructure;

14. improving roadside drainage;

15. widening an existing ramp;
16. flattening a horizontal or vertical curve; or

17. increasing the vertical clearance at an underpass.

Chapter 54 provides the criteria for the design of a 3R freeway project.

**40-6.01(06) 3R Project, Non-Freeway**

A 3R project (rehabilitation, restoration, resurfacing) on an existing non-freeway is intended to extend the service life of the existing facility and to enhance highway safety. A 3R project should make cost-effective improvements to the existing geometrics, where practical. A 3R project on the mainline or at an intersection is work on the existing alignment. Minimal right of way acquisition may be required. Improvements include the following:

1. pavement resurfacing or rehabilitation or a limited amount of pavement reconstruction (30% or less of the traveled way area);

2. bridge rehabilitation or replacement;

3. lane or shoulder widening;

4. upgrading the structural strength of shoulders;

5. flattening a horizontal or vertical curve;

6. adjustment to the roadside clear zone;

7. flattening side slopes;

8. converting an existing median to a 2-way left-turn lane;

9. adding a truck-climbing lane;

10. converting an uncurbed urban street into a curbed street;

11. revising the location, spacing, or design of an existing drive along the mainline;

12. adding or removing a parking lane;
13. bridge widening and associated substructure work to accommodate the widening;

14. bridge railing upgrading or replacement;

15. bridge-deck overlay;

16. work to preserve the bridge substructure;

17. adding sidewalks;

18. relocating utility poles;

19. upgrading guardrail or other safety appurtenances to satisfy current criteria;

20. other geometric or safety improvements to an existing bridge;

21. drainage improvements;

22. increasing vertical clearance at an underpass;

23. intersection improvement (e.g., adding turn lanes, flattening turning radii, channelization, corner sight-distance improvements, etc.);

24. adding new or upgrading traffic signals; or

25. other spot improvements.

Specifically related to the level of pavement improvement, the following definitions apply.

1. Resurfacing. Resurfacing consists of the placement of additional surface material over the existing restored or rehabilitated roadway or structure to improve serviceability or to provide additional strength.

2. Restoration or Rehabilitation. Restoration or rehabilitation is defined as work required to return the existing pavement to a condition of adequate structural support or to a condition adequate for the placement of an additional stage of construction. This can include milling the existing pavement.

Chapter 55 provides the criteria for the design of a 3R non-freeway project.
40-6.01(07) Partial 3R Project

A partial 3R project is intended to extend the service life of the pavement and, where practical, to enhance highway safety. Geometric design improvements are included to correct obvious deficiencies on the existing highway. Right of way acquisition is rarely involved. Partial 3R improvements include the following:

1. pavement resurfacing;
2. lane or shoulder widening;
3. adjustments to the roadside clear zone;
4. relocating utility poles;
5. upgrading guardrail or other safety appurtenances to satisfy current criteria;
6. correcting a high-accident locations;
7. drainage improvements; or
8. improving superelevation to the extent practical.

Chapter 56 provides the criteria for the design of a partial 3R project. The only partial 3R treatment permitted on an NHS route is preventative maintenance. All types of partial 3R treatments are permitted on a non-NHS route. Chapter 304 provides pavement-design criteria for each type of project.

40-6.01(08) High-Accident-Location Improvement, Non-Freeways

1. **Non-NHS Route.** This type of project is intended to make improvements to correct a safety problem at a location that is identified through the FHWA-approved INDOT Safety Improvement Program process, which applies to either a State or local facility. It is not intended to provide a general upgrading of the highway, as is a project categorized as new construction or reconstruction, or 3R. No specific design criteria for this type of improvement are described herein. The objective is to rapidly correct an identified accident hazard using the highest level of design criteria as practical at the site considering existing site limitations (e.g., right-of-way restrictions).

2. **NHS Route.** A high-accident-location improvement must satisfy the appropriate criteria, or a design exception must be obtained. This is also identified through the FHWA-approved INDOT Safety Improvement Process. However, the design criteria to be used are those for new construction or reconstruction, or 3R, based on the criteria described in Section 40-6.02(01).
40-6.01(09) Traffic-Control-Devices Project

A traffic-control-devices project is programmed specifically to install, replace, or remove signs, pavement markings, traffic signals, highway lighting, etc. No other work is included, except that a traffic-signal project will include curb ramps at each involved intersection. Part VII provides the criteria for the installation of traffic-control devices on a freeway or a non-freeway.

40-6.02 Application

40-6.02(01) National Highway System (NHS) Project

For long-range transportation planning purposes, INDOT has evaluated the State highway system to determine which routes warrant reconstruction or 4R, and which routes warrant a 3R-type improvement. Figure 40-6A provides a map of the State highway system to indicate 3R and 4R routes. The project scope of work definitions in Section 40-6.01 will apply to each project on the NHS. The following will apply to the use of Figure 40-6A for a 3R or 4R route on the NHS.

1. General. The factors that determine if a project should be classified as 3R or 4R are as follows.

   a. If 70% or more of the existing traveled-way pavement area can be retained and resurfaced, the project can be classified as 3R. If not, the project should be classified as 4R.

   b. An assessment of the level of service (LOS) for the 10-year traffic volume projection can determine if the project is 3R or 4R, based upon the expected service life of the pavement.

Other factors should also be considered in making the project scope of work determination (e.g., accident rates).

2. Freeway. A freeway project will be classified as new construction, complete reconstruction, partial reconstruction, or 3R. See Section 40-6.01 for definitions.

3. 4R Non-Freeway Route. Environmental Policy Team or the local jurisdictional agency will determine the level of service (LOS) for the 10-year traffic volume projections based on the discussion in Section 40-2.0. If this is LOS of D or better, it will be acceptable to design the project using the 3R geometric-design criteria shown in Chapter 55. If the...
projected LOS will not satisfy D, the facility will be designed according to the criteria for new construction or reconstruction. A bridge replacement, bridge-deck replacement, or bridge widening should be designed to satisfy 4R criteria.

4. **3R Non-Freeway Route.** The project will be designed according to the 3R geometric-design criteria shown in Chapter 55. However, consideration should be given to using the 4R criteria.

5. **3R Project.** If the 3R project scope of work is selected, costly items (e.g., bridge reconstruction or replacement, alignment corrections), which have a long service life and can be incorporated into a future 4R project, should be constructed to satisfy 4R design criteria as part of the 3R project.

6. **Combination Project.** If a project will include both 3R and 4R work, the overall project scope of work classification should be based on the predominant type of work. For example, a 6-mi resurfacing project which includes the replacement of a mainline bridge to 4R criteria should be classified as a 3R project, unless the bridge is considered to be a major structure and its replacement cost is equal to or greater than that of the 3R roadway work.

7. **S-Line.** Each S-line should be individually evaluated to determine the appropriate design criteria (4R or 3R) based on the factors described herein if it is on the NHS, or Section 40-6.02(02) if it is not on the NHS.

   If an S-line is designed to 3R criteria, the intersection sight distance must be determined based on the 4R criteria described in Section 46-10.0.

The requirements described herein must be used to design each NHS project regardless of the funding source, whether Federal, State, or local-agency funds are used. However, the values shown in the AASHTO Policy on Geometric Design of Highways and Streets may be used as minimum values if they are lower than similar values shown herein where restricted conditions warrant.

**40-6.02(02) Non-NHS Project**

The project scope of work definitions in Sections 40-6.01 and 40-6.02(01) and Figure 40-6A are intended only as guidance for a non-NHS project. The decision of classifying a project that is not on the NHS should be made based on the future plans of the jurisdictional highway agency for the entire route between logical termini for the foreseeable future (20 years). The future
plans for a route must consider current and projected traffic volumes, anticipated land use, and accident experience. The following are examples of applying this concept to a non-NHS project.

1. **Example One.** Approximately 60% of the pavement on a 6-mi section of a county road will be replaced. The remainder of the pavement is in reasonably good condition and requires only milling and resurfacing. The 6-mi section is part of a 30-mi county route which is the main highway between two small towns. The existing road has a LOS of A, and it is anticipated to provide a LOS of B based on 20-year projected traffic volume. There is no adverse accident experience for the last three years. Based on this information, a highway agency could decide to designate the 3R classification and construct the road to 3R design criteria. This is acceptable, though more than 30% of the pavement is being completely replaced.

2. **Example Two.** Approximately 40% of the pavement on a 6-mi section of county road will be replaced. The remainder of the segment will be resurfaced. This segment of road is part of a 25-mi county route which connects two small towns. This county road is located approximately 20 mi from a major metropolitan area. It is anticipated that, within the next 20 years, there will be considerable residential and commercial development adjacent to this portion of the county road because of its proximity to the rapidly-expanding metropolitan area. The current LOS is B, but projected traffic volume indicates that the LOS will drop to D in 10 years and to F in 20 years. The highway agency has two options. It could decide to design the project to 3R criteria for the present and, then, undertake a 4R project in 10 years once the pavement is likely in need of major work. Its second option is to construct the project to 4R criteria now to satisfy future traffic demands.

3. **Example Three.** A 6-mi section of highway, which is located on INDOT’s 3R highway system, requires complete pavement replacement because of poor drainage. The Central Office has rechecked the status of this highway with the district office and has verified that there are no plans for work on the remainder of this route in the future (20 years) except for 3R-type work. The current LOS is B, and it is anticipated to remain at B for the next 20 years. There is no adverse accident experience and no anticipated major land development along the route. INDOT can decide to design the project to 3R design criteria, even though all pavement is being replaced.

4. **Example Four.** A 200-ft bridge on the State’s 3R system requires complete replacement. There are sharp horizontal curves on each end of the bridge where numerous accidents have occurred during the last three years. It has been decided to correct the poor alignment on the bridge approaches and to construct the approaches and bridge on a new location. The total length of the project is 1.5 mi. The Central Office has discussed the
status of this site with the district office, and both have agreed that it should remain on the 3R system. The current LOS is B, and it is estimated that the LOS will be C in 20 years. There are no plans except to perform 3R-type work to the remainder of the road in the future (20 years). INDOT can decide to design the entire project to 3R criteria.

5. **Example Five.** A 6-mi segment of a route on INDOT’s 3R system requires replacing 20% of the pavement and resurfacing the remaining 80%. The current LOS is D and will deteriorate to E in 5 years. There is rapid residential, commercial, and industrial development in the area. The Central Office and the district office agree that the entire route was properly classified as a 3R route. However, this one 6-mi segment is an exception because rapid growth adjacent to it is expected to occur. The appropriate solution is to upgrade the facility to accommodate anticipated traffic demand for the next 20 years and to design the project to 4R criteria.

40-6.02(03) Procedures

For an INDOT project, the scope of work is selected based on the following procedure.

1. The district office initially identifies the project scope.

2. The project is programmed based on the project scope determined by the district.

3. The Office of Environmental Services will make the final decision on the scope of work. However, for each NHS project which has an estimated construction cost exceeding $1 million, FHWA will meet with representatives of the Office of Environmental Services to cooperatively agree on the project classification and whether it should not be exempt from FHWA oversight. This will occur as early in the project scoping process as possible, so that FHWA can have input on each project which is subject to its oversight. The meeting will be held as soon as an initial concept for the project design has been developed. The results, including classification and oversight determination, will be documented in the Engineer’s Report. The cover of the report will indicate whether the project is exempt or not exempt from FHWA oversight.

4. The Production Management Division, during project design, may re-evaluate the project scope and request the Office of Environmental Services to modify the scope of work.

For a Federal-aid project not on the State highway system, the project scope of work determination will be based on the future plans of the local agency for improvements to its local road or street system. The philosophy provided in Section 55-2.01(02) Item 2 should be applied
to a local project. The local agency must submit a letter to the Planning Division to document the local agency’s plans for that facility in the foreseeable future.

If the project is on the NHS and the estimated construction cost exceeds $1 million, the Planning Division will schedule a meeting with the local agency and FHWA to agree upon a project classification (3R or 4R). This meeting should occur early in the scoping process so that FHWA can have input on each project that is subject to its oversight.

### 40-7.0 FHWA INVOLVEMENT

The 1991 *Intermodal Surface Transportation Efficiency Act* (ISTEA), and the *National Highway System Act of 1995*, in addition to a realignment of the Federal-aid system, revised the role of the Federal Highway Administration for each project. The *Transportation Efficiency Act for the 21st Century (TEA-21)* of 1998, further revised the role of FHWA for each project as described below.

1. **Highway System.** FHWA oversight is required only on an Interstate-route project.

2. **Project Scope of Work.** FHWA oversight is required only on a new Interstate-route construction or reconstruction project.

3. **Project Cost.** FHWA oversight is required only on an Interstate-route project with an estimated construction cost exceeding $1 million.

The jointly-approved INDOT and FHWA *Stewardship and Oversight Agreement* provides the basis for the stewardship and oversight of FHWA for the use of Federal-aid funds by INDOT.

For a project with INDOT oversight, FHWA will not be involved with day-to-day project activities, including field reviews, design approval, public-hearing certification, design exceptions, PS&E submittal, etc. Such project is still subject to the FHWA Program and Process Review. However, each Federally-funded project will be designed in accordance with the appropriate criteria described herein, and the INDOT *Standard Specifications* and *Standard Drawings*, regardless of FHWA review.

The FHWA is not precluded from reviewing or investigating a phase of the Federal-aid program including control documents or a Federal-aid project, especially a project including a unique feature or unusual circumstance such as a special structure design, experimental feature, etc., for which it is desirable to have FHWA oversight. The oversight determination for each such
project will be made at the meeting discussed in Section 40-6.02(03), Item 3. INDOT and FHWA will meet to determine oversight responsibility as shown in Figure 40-7A.

40-8.0 ADHERENCE TO DESIGN CRITERIA [REV. FEB. 2014, JUL. 2014, MAR. 2016]

40-8.01 Department Intent [Rev. Jul. 2014]

The Department’s intent is that all geometric design criteria described in this Part should be satisfied. This is intended to ensure that the Department will provide a highway system which satisfies the transportation needs of the State and provides a reasonable level of safety, comfort, and convenience for the traveling public.

Chapters 40 through 56 provide information on geometric design for application to each individual project. The values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values without a design exception if they are lower than similar values shown herein. See Chapters 53 and 55 for specific exceptions.

A roadway functionally classified as collector or local road that has an average daily traffic volume of 400 vehicles per day or less may be designed using AASHTO’s *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)*.

AASHTO’s *A Policy on Design Standards, Interstate System* is the minimum design criteria for interstates.

The designer is responsible for satisfying these criteria in the project design. However, this will not always be practical. In addition to crash history, designers should consider cross-section consistency as well as reasonable consistency in geometric alignment and sight distance along the corridor when considering a design exception. The Department’s procedures for identifying, justifying, and processing exceptions to the geometric design criteria shown in chapters 40 through 56 are described below.

40-8.02 Hierarchy of Design Criteria

40-8.02(01) Level One [Rev. Feb. 2014]

Level One controlling design criteria are those highway design elements which are judged to be the most critical indicators of a highway’s safety and its overall serviceability. Not all of the design information described in this Part qualifies as a Level One criterion. The Department and
FHWA have identified the following design elements as Level One. The formal documentation and approval process for a design exception or waiver described in Section 40-8.04 must be followed if these criteria are not satisfied.

1. design speed for mainline or interchange ramp *
2. lane width
3. shoulder width for uncurbed section or curb offset for curbed section
4. bridge width for new, rehabilitated, or existing bridge to remain in place
5. structural capacity for new, rehabilitated, or existing bridge to remain in place
6. horizontal curvature, i.e., minimum radius
7. superelevation-transition length
8. application of stopping sight distance to a horizontal curve or a vertical curve
9. maximum grade
10. travel-lane cross slope
11. superelevation rate
12. minimum vertical clearance
13. *Americans with Disabilities Act (ADA) compliance**; and
14. bridge-railing safety performance criteria.

* An exception to design speed is not allowed. Instead, the designer will use the Department’s applicable criteria for the project design speed and will, if needed, seek an exception to each individual design element which does not satisfy the design-speed requirement, e.g., a horizontal or vertical curve.

** Requires a determination of technical infeasibility or technical inquiry. See Section 40-8.04(01).

It is not necessary to submit a Level One checklist for an S-line that does not exceed the work necessary to build the appropriate public-road approach, including the required taper distance to account for transitioning to the existing pavement width. This requirement does not relieve the designer of having the S-line satisfy all critical design elements in the area, i.e., maximum grade, vertical stopping sight distance, and intersection sight distance.

The existing minimum vertical clearance dimension for a structure carrying a roadway over a railroad should be field-measured. Standard track maintenance procedures performed by a railroad company often result in an increase in the rail elevation. Therefore, the minimum vertical clearance dimension shown on prior construction plans will not be a true indication of the current minimum vertical clearance. Each report or plan identifying the existing minimum vertical clearance dimension over a railroad should indicate the date of the field measurement. This
dimension should be shown on the profile view of the General Plan sheet with a corresponding note identifying the date of the field measurement.

Each Level One criterion must be satisfied for the entire project length, including all paving exceptions. If a criterion is not satisfied, the designer must apply for a design exception or revise the plans.

The Level One Criteria Checklist is to be included with each submittal. If there are no changes to the plans from the previous submittal that affect the Level One criteria, it is permissible to copy the previous Checklist form and add a comment. The comment should indicate that there are no changes to the plans that affect Level One criteria. Such statement should be initialed and dated for the current submittal. A completed Limited Review Certification should be submitted at the Final Check Prints and Final Tracings stages. These forms are available at www.in.gov/dot/div/contracts/design/dmforms.

40-8.02(02) Level Two

Level Two design criteria are those which are judged to be important indicators of a highway’s safety and serviceability, but are not considered as critical as the Level One criteria. If a Level Two criterion is not satisfied, the designer will document in the project file that the criterion has not been satisfied and will provide a brief rationale for not satisfying it. However, it is not necessary to prepare an in-depth documentation to justify the decision.

The brief rationale for a project’s in accordance with the intersection sight distance requirements should include the following:

1. design speed;

2. summarization of accident data for the most recent available 3-year period;

3. evaluation of the accident data which is related to intersection sight distance; and

4. approximate cost of accordance with the intersection sight distance requirements.

For a local-agency project, the local agency should furnish written concurrence with a decision not to improve the intersection sight distance to full accordance with the requirements. This concurrence may be in the form of a local elected official signing off on the Level Two design exception, or a separate letter from the elected official.
The Level Two design criteria are as follows:

1. all roadside-safety design elements (see Chapter 49);
2. obstruction-free zone for a non-freeway 3R project;
3. median- or side-slope rate;
4. access control for a freeway or a freeway ramp intersection with a non-freeway facility;
5. intersection sight distance;
6. freeway acceleration lane length;
7. freeway deceleration lane length;
8. median width;
9. shoulder cross slope and rollover;
10. auxiliary lane or shoulder width on a non-freeway;
11. minimum grade for drainage;
12. minimum level-of-service criteria;
13. parking-lane width;
14. two-way left-turn lane width;
15. critical length of grade; and
16. truck SSD for specific application (see Section 42-1.0).

40-8.02(03) Level Three
Level Three includes the design criteria not listed in Level One or Two. No action is required if a Level Three criterion is not satisfied. However, the designer should informally notify his or her supervisor of the situation.

40-8.03 Design-Exception Process

The design-exception process will be applied as follows:

1. **Project Scope of Work.** The design exception process will apply to a new construction, reconstruction (4R), 3R or partial-reconstruction freeway, or 3R non-freeway project. The application of the design-exception process to a partial 3R project is discussed in Chapter 56. The design-exception requirements do not apply to a high-accident-location project on a non-NHS route because there are no specific design criteria. The design-exception process does not apply to a signing, pavement markings, traffic-signal installation, or traffic-barrier project which requires little or no roadway work.

   The design-exception process does not apply to a preventative maintenance project on the National Highway System. An exception is not required for the retention of an existing feature which does not satisfy INDOT criteria. In effect, INDOT is maintaining the project as built and as agreed to with FHWA in the project agreement. However, a new design feature which does not satisfy INDOT criteria created by the project, or existing ones made worse, must be addressed in an exception, because such action in effect changes the project as built. Preventative maintenance includes restoration and rehabilitation of specific elements of a highway facility if it can be demonstrated that such activities are a cost-effective means of extending the service life of the existing facility. Pavement preventative maintenance treatments are discussed in Section 304-6.04. Bridge preventative maintenance treatments are discussed in Section 72-1.04.

2. **Federal-Aid Project on the National Highway System.** A design element that does not satisfy the Level One criteria will be addressed as described in Section 40-8.04. For a Level Two design exception, the designer should inform FHWA of the exception if the project is not exempt from FHWA oversight.

3. **State-Funded Project, FHWA-Oversight Exempt Project, or Project Not on the NHS.** Each design element that does not satisfy the Level One criteria should be documented and approved as an INDOT exception. If a Level Two criterion will not be satisfied, the designer should document in the project file that the criterion has not been satisfied, and should provide a brief rationale.
4. **Locally-Funded Project.** The designer should document where the proposed design deviates from the criteria provided in this Part.

5. **Signing and Dating the Design-Exception Request.** For a Level One or Level Two design-exception request, the designer should sign and date the request. A consultant, if used, should also include the name of the consulting firm below the signer’s name.


The designer will not request an exception to the Level One design criteria until he/she has fully evaluated the impacts of the proposed design (i.e., the exception) and the associated impacts of fully satisfying the Level One criteria. The evaluation process should include obtaining comments from the applicable Divisions including the following:

1. Traffic Engineering Division;
2. Asset Management Division;
3. Pavement Division, Office of Geotechnical Engineering;
4. Real Estate Division
5. Utilities and Railroads Division;
6. Bridges Division, Office of Hydraulics;

After review by the applicable offices or teams, the design exception should then be routed in the order shown below for further comments, recommendations, and final action.


Each design element that does not satisfy the Level One controlling criteria will require a written design exception. This includes all paving exceptions, S-lines, and traffic maintenance phases. Multiple design elements may be included in a single design exception document; however, each design element must have its own approval cover letter. An editable version of the design exception cover letter, Figure 40-8C, is available on the Department’s website, at [www.in.gov/dot/div/contracts/design/dmforms/](http://www.in.gov/dot/div/contracts/design/dmforms/).

A design exception for a local-agency project or a State project involving an element on a local agency’s road must be signed by the local elected officials who have jurisdiction of the project or road prior to routing for review.
For new construction, reconstruction (4R) and 3R or partial-reconstruction (4R) freeway projects the design exception requirements are described in item 1.

For 3R Non-Freeway projects a streamlined design exception is used. The design exception requirements are described in item 2. The streamlined design exception is intended to document the satisfactory performance of existing design features. Retaining or replacing existing geometric design features in-kind may be appropriate when satisfactory performance is documented.

For all projects, an exception to the requirements of the Americans with Disabilities Act (ADA) requires a determination of technical infeasibility as described in item 3 below.

The design exception request must contain all of the necessary information or references without requiring the reviewer to obtain additional information (e.g., plan sheets, copies of pages of this Manual that pertain to the design exception request, or copies of pertinent pages of the AASHTO Policy on Geometric Design of Highways and Streets.)

1. **Level One Criteria Design Exception for 4R, Partial 4R Freeway, and 3R Freeway project.** The design exception will, at a minimum, address the following.

   a. Project Description. This includes project location, functional classification, description of work, and type of area (residential, commercial, rural, etc.) in which the project is located. The location of the design exception should be identified by referencing it to the nearest Department-maintained route or other major point such as a county line.

   b. Design Feature. This is a description of the design feature that does not satisfy the Department’s criteria. Both the proposed criterion and the Department criterion should be identified, with respective design speeds where applicable. Drawings should be used to explain the criterion if necessary. The reason for the design exception request should be indicated.

   c. Construction Costs. This is the additional cost to construct the feature to satisfy the Department criterion. An abbreviated breakdown of the costs should be included.

   d. Project Design. This includes the basic design parameters of the project (e.g., current and projected 20-year traffic volumes, design speed, posted speed, percent trucks, design criteria, terrain, and access control).
e. Crash Analysis. In addition to furnishing the summary of crash history for the previous 3-year period, the crash data must be presented as follows. For a new roadway, see item h., Safety.

1) It should be summarized and described in general terms (e.g., type, severity, contributing circumstances).

2) All available sources (city, county, and state police) must be contacted to obtain the data and be identified in the design exception request. For INDOT projects, crash history is available through the Automated Reporting Information Exchange System (ARIES), which is the Web portal to the Indiana Vehicle Crash Report System database maintained by the Indiana State Police.

3) The crash experience which is related to the design feature and does not satisfy Department criteria should be analyzed and evaluated. The evaluation may include, for example, a comparison of the crash rate on the highway to the Statewide rate for that type of facility or may include a statistical analysis of the crash experience at the location of the design feature (e.g., a horizontal curve).

   The Road Hazard Analysis Tool (RoadHAT) Form 1 can be used to develop the comparison between similar facility types. The RoadHAT program is available from the INDOT Technical Application Pathway (ITAP).

f. Cost-Effective Analysis. A cost-effective analysis should be conducted to justify the proposed design exception, if applicable (e.g., there are crashes related to the design feature in question. See Chapter 50 for more information.

g. Ancillary Impact. Adverse effects that the design exception will have on other design elements on or near the project site must be evaluated and documented (e.g., sight distance on a horizontal curve impacts intersection sight distance at an intersection outside the project limits).

h. Safety. The safety impacts of the design exception must be evaluated and documented. For example, if there were no crashes with the existing condition in place and the project will match or improve the situation, the conclusion is that there is no increase in crashes. For a new roadway (i.e. no crash history), the
safety impacts can be evaluated by comparing the predicted number of crashes using the proposed value for the design element to predicted crashes using the value that satisfies the controlling criterion requirement.

i. Mitigation. The designer must document the proposed mitigation measures which will be implemented to alleviate the retention or construction of the design feature which does not satisfy Department criteria (e.g., traffic-control devices). Mitigation resources are available from the FHWA publication *Mitigation Strategies for Design Exceptions*. This publication is available from the FHWA website at [http://safety.fhwa.dot.gov/geometric/pubs/](http://safety.fhwa.dot.gov/geometric/pubs/).

j. Other Factors. Other factors which may have an effect on the final recommendation should be discussed. For example, the following:

1) projected service life of the facility after construction is completed;
2) compatibility with adjacent sections of the proposed project;
3) probable time before reconstruction of the section is anticipated; and
4) environmental and right-of-way impacts of satisfying the Department criteria.

2. **Level One Criteria Design Exception for 3R Non-Freeway Projects.** A streamlined design exception may be used to retain or replace an existing geometric feature in-kind or when the proposed criteria improve the existing but do not meet the criteria found in Chapter 55. When multiple design exceptions are required for a 3R Non-Freeway project, a single document with multiple cover sheets should be created. At a minimum the design exception will include the following.

a. **Project Description.** Include the project location, functional classification, description of work, design year ADT including the percentage of trucks, and type of area (residential, commercial, rural, etc.) in which the project is located.

b. **Design Feature.** Include a description of the design feature that does not satisfy the criteria in Chapter 55. The existing criterion, the proposed criterion and the criterion in Chapter 55 should be identified, with respective design speeds where applicable. Drawings should be used to explain the criterion if necessary. The reason for the design exception request should be indicated. The intent to retain an existing geometric condition or replace it in-kind should be clearly stated.
c. Crash Analysis. Using the most recent 3-year crash history, document that the roadway is performing as expected. For INDOT projects crash history is available through the Automated Reporting Information Exchange System (ARIES), which is the Web portal to the Indiana Vehicle Crash Report System database maintained by the Indiana State Police.

An acceptable crash history may be no crashes, an evaluation using RoadHAT Form 1 resulting in an ICF and ICC of 0 or less, or a review of crash data that indicates there is not an apparent relationship between existing roadway geometry or operation (e.g. sharp horizontal curve, lack of exclusive left turn lane) and crash location and manner of collision (e.g. head-on, rear end, right angle). The RoadHAT program is available from the INDOT Technical Application Pathway (ITAP).

A summary of the raw data including the following should be included in tabular form at a minimum: year, location, manner of collision, and severity level (e.g. property damage only, injury, or fatal).

d. Plans for Expansion. Document that roadway expansion is not planned due to increased traffic demand or as part of an overall corridor improvement. For the State Highway System, the district Technical Services Division, in cooperation with the central office Asset Management Division Office of Technical Planning can provide this information.

e. Compatibility with Adjacent Sections. Indicate if the proposed roadway cross section is compatible with the roadway section before and after the project limits, i.e. the same cross section width or negligibly wider or narrower than the adjacent roadway. In general, the proposed roadway should not be narrower than the existing roadway. Treatment of an existing roadway section that is wider than the adjacent sections should be addressed on a project-by-project basis.

f. Mitigation. The designer must document the proposed mitigation measures which will be implemented to alleviate the retention or construction of the design feature which does not satisfy minimum criteria. Mitigation resources are available from the FHWA publication Mitigation Strategies for Design Exceptions. This publication is available from the FHWA website at http://safety.fhwa.dot.gov/geometric/pubs/.
3. **ADA Compliance.** When an element of a pedestrian access route (PAR) cannot be constructed in full compliance with the ADA standards, one of the following must be submitted:

a. **Technical Infeasibility Request:** A technical infeasibility request should be submitted when an element of the PAR cannot fully comply due to an existing constraint that cannot be removed or adjusted, e.g. a building. This type of request should be rare for new and reconstruction projects, but may be applicable to a resurface or other alteration project.

**Technical Infeasibility Request Example:** As part of a resurface project, a non-compliant curb ramp is located at an intersection that is constrained by a building designated as historic. The existing curb ramp does not contain a turning space and the running slope of the ramp is greater than 8.33%. The building location is such that only a non-compliant turning space can be constructed and the running slope cannot be reduced without impacting the building. A technical infeasibility requested should be submitted for review.

In this case, compliance is technically infeasible. Compliance is only required to the extent that it does not threaten or destroy the historic feature. The approved technical infeasibility request should be filed with the project coordination files and with the Level One computations. The element will be removed from the owner’s transition plan inventory list.

b. **Technical Inquiry:** A technical inquiry should be submitted when an existing physical constraint makes it impractical, within the scope of work, for an element of the PAR to fully comply. This type of request is most commonly associated with resurface or other alteration projects where constructing the element to full compliance falls outside the scope of work.

For all projects, a technical inquiry may be submitted for an ADA question, clarification on an ADA policy, or best practice proposal.

**Technical Inquiry Example:** As part of a resurface project, a non-compliant curb ramp is located at an intersection that is constrained by right of way and utilities. The existing curb ramp does not contain a turning space and the grade of the ramp is greater than 8.33%. The right of way limits and utility locations are such that only a non-compliant turning space can be constructed and the ramp running slope can be lessened but not made fully compliant. Full compliance would require the acquisition of right of way and the relocation of utilities, which are not
part of the scope of work. A technical inquiry request should be submitted for review.

The curb ramp should be made compliant to the maximum extent practical. The approved technical inquiry should be filed with the project coordination files and with the Level One computations. The element will remain on the owner’s transition plan inventory list to be addressed by a future project.

A determination of technical infeasibility and technical inquiry does not constitute a waiver of the ADA requirements, but rather serves as a process of sufficiently documenting alternatives considered, existing constraints, and costs associated with compliance for later use, if necessary, as the basis for a defense regarding a complaint or litigation.

The Department’s ADA Committee will review requests in accordance with the Technical Infeasibility Policy. The Committee will review requests for determination of technical infeasibility and inquiry for projects that contain federal-aid funds or are 100% State-funded. The determination of technical infeasibility and technical inquiries are the responsibility of the Local Public Agency (LPA) for 100% locally-funded projects.

A request for determination of technical infeasibility or inquiry should be sent to the Director of Highway Design & Technical Services. In addition, the Title VI/ADA Program Manager must receive a copy of the request. The request submission should include the following:

a. DES Number, if available;
b. project location and description of the scope of the project;
c. a detailed explanation of the element and ADA standard that cannot be met.
d. a detailed explanation of why the standard cannot be met;
e. (For technical infeasibility requests only) a detailed explanation of at least two options considered before requesting a determination of technical infeasibility and why these options were not pursued further;
f. a recommendation for a proposed solution. This should include an explanation why the proposed solution is the best fit for the given circumstances and how it provides accessibility to the maximum extent feasible;
g. an itemization of the costs to construct the element compliantly and comparison to the overall project cost; and
h. pictures and/or drawings of the actual project location and proposed solutions.
40-8.04(02) FHWA Procedure

A proposed exception to the Level One criteria for a project on the Interstate system and has FHWA oversight must be submitted to the FHWA Indiana Division’s Administrator for review and approval. A proposed exception for a Federal-aid project will not be submitted to FHWA until after the exception has completed the internal Department process; see Section 40-8.04(01). The documentation required for the Department’s exception process will be sufficient for FHWA evaluation.

For a Level Two design exception, the designer should inform FHWA of such exception on an FHWA oversight Interstate-system project.


The Bridges director can only take approval action on a design exception to reduce or retain the existing vertical clearance over the Interstate system that is less than the required 16'-0” after coordinating formally with the Department of Defense (DOD), Surface Deployment and Distribution Command Transportation Engineering Agency (SDDCTEA). This coordination is necessary whether the work is a new construction project, a project that does not provide for correction of an existing substandard condition, or a project that creates a substandard condition at an existing structure. The requirement to provide or preserve the 16’-0” vertical clearance extends to the full roadway width including shoulders for the through lanes, as well as to ramps or collector-distributor roadways in an Interstate-to-Interstate interchange. This requirement applies to the Indiana Toll Road since it is part of the Interstate System.

The designer must include the completed Interstate Vertical Clearance Exception Coordination form with the design exception request. In addition to the design exception information in Section 40-8.04(01), item 1, the submission should include preliminary plan and profile sheets for both the Interstate highway and the overpassing structure. The Vertical Clearance Exception Coordination form is available at www.in.gov/dot/div/contracts/design/dmforms.

The Bridges director will coordinate directly with DOD. A response time of 30 days after being sent from INDOT should allow an adequate review period for the SDDCTEA.

If the SDDCTEA reply does not agree with the design exception, INDOT personnel should consider feasible mitigation measures and should notify the SDDCTEA of the proposed action.
On a project with FHWA oversight, INDOT personnel should work jointly with FHWA in determining proposed mitigation measures.

Coordination with the SDDCTEA is to be completed before transmitting the design exception to the FHWA for a project with FHWA oversight. The submission to the FHWA should include documentation that the coordination with the SDDCTEA has been satisfactorily completed.

**40-8.04(04) Procedure for Local Project with Federal Funds**

For a local project with Federal funds, a design-exception request will follow the procedures in Section 40-8.04(01). The design exception must be signed by the local elected officials who have jurisdiction of the project or road prior to routing for review.

**40-8.04(05) Procedure for 100% Locally-Funded Project**

For a project funded entirely with local funds, the local agency should establish a procedure so that an individual with the proper authority will approve the design exception.

**40-8.04(06) Signature Block**

The Bridges or Highway Design and Technical Support Division director is responsible for the approval of each proposed exception to the Level One criteria for an FHWA-exempt project on the NHS system, or each Federal-aid project on a non-NHS route. The Division director must also approve each design exception for a Federal-aid project on the NHS that is not exempt from FHWA oversight before the exception is submitted to FHWA for its approval. The Division director is responsible for approval of each design-exception request for a 100% State-funded project.

**40-8.05 Documentation**

The Level One Controlling Criteria Checklist should be used to document the project’s accordance with the Department’s Level One design criteria. An editable version of this form is available on the Department’s website at [www.in.gov/dot/div/contracts/design/dmforms/](http://www.in.gov/dot/div/contracts/design/dmforms/), Checklist 40-8B. This applies to each project, with or without design exception. The designer should complete the appropriate boxes on the form. The determination of whether or not the proposed project design satisfies the INDOT design criteria is dependent upon the project scope of work and the design criteria described herein. If, for example, a 3R non-freeway project is being designed, Chapter 55 will apply.
<table>
<thead>
<tr>
<th>PROJECT SCOPE OF WORK</th>
<th>DESIGN YEAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Construction or Reconstruction</td>
<td>20</td>
</tr>
<tr>
<td>3R, Freeway</td>
<td>20 *</td>
</tr>
<tr>
<td>3R, Non-Freeway</td>
<td>20 *</td>
</tr>
<tr>
<td>Partial 3R</td>
<td>10</td>
</tr>
<tr>
<td>Intersection Improvement</td>
<td>20 *</td>
</tr>
</tbody>
</table>

*Note: The design year is the number of years after the work is expected to be completed.*

*For a partial 3R project, this may be 10.*

**RECOMMENDED DESIGN-YEAR TRAFFIC VOLUME FOR ROAD DESIGN**

*Figure 40-2A*
<table>
<thead>
<tr>
<th>TYPE OF FACILITY</th>
<th>MEASURE OF EFFECTIVENESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway</td>
<td>Density (passenger cars per mile per lane)</td>
</tr>
<tr>
<td></td>
<td>Average travel speed (mph)</td>
</tr>
<tr>
<td></td>
<td>Flow rate (passenger cars per hour)</td>
</tr>
<tr>
<td>Basic freeway segment</td>
<td></td>
</tr>
<tr>
<td>Weaving area</td>
<td></td>
</tr>
<tr>
<td>Ramp junction</td>
<td></td>
</tr>
<tr>
<td>Highway of 4 or More Lanes</td>
<td>Density (passenger cars per mile per lane)</td>
</tr>
<tr>
<td>Two-Lane Highway</td>
<td>Time delay (%)</td>
</tr>
<tr>
<td></td>
<td>Average travel speed (mph)</td>
</tr>
<tr>
<td>Arterial</td>
<td>Average travel speed (mph)</td>
</tr>
<tr>
<td>Signalized Intersection</td>
<td>Average individual stopped delay (seconds/vehicle)</td>
</tr>
<tr>
<td>Unsignalized Intersection</td>
<td>Reserve capacity (passenger cars per hour)</td>
</tr>
</tbody>
</table>

**MEASURE OF EFFECTIVENESS FOR LEVEL OF SERVICE**

*Figure 40-2B*
<table>
<thead>
<tr>
<th>Geographic Location</th>
<th>System</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural</td>
<td>State Highway</td>
<td>n/a</td>
<td>55 mph *</td>
</tr>
<tr>
<td></td>
<td>Non-State Highway</td>
<td>30 mph</td>
<td>55 mph</td>
</tr>
<tr>
<td>Urban</td>
<td>State Highway</td>
<td>n/a</td>
<td>30 mph</td>
</tr>
<tr>
<td></td>
<td>Non-State Highway</td>
<td>20 mph</td>
<td>55 mph (day)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50 mph (night)</td>
</tr>
</tbody>
</table>

**Notes:**

* 60 mph for a facility of 4 or more lanes

1. *This table applies to a non-Interstate facility.*

2. *See Section 40-3.02(03) for exceptions.*

**LEGAL SPEED LIMITS**

*Figure 40-3A*
<table>
<thead>
<tr>
<th>DESIGN VEHICLE TYPE</th>
<th>SYMBOL</th>
<th>DIMENSIONS (ft)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Overall</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Height Width Lgh.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>Passenger Car</td>
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* The Indiana Design Vehicle (IDV) is equivalent to the WB-65.
  WB1, WB2, WB3, and WB4 are effective vehicle wheelbases.
  S = Distance from the rear effective axle to the hitch point.
  T = Distance from the hitch point to the lead effective axle of the following unit.

a = This is overhang from the back axle of the tandem axle assembly.
b = Combined dimension is 19.4 ft and articulating section is 4 ft wide.
c = Combined dimension is typically 10.0 ft.
d = Combined dimension is typically 12.5 ft.
e = Dimensions are for a 150-200 hp tractor excluding wagon length.
f = To obtain the total length of tractor and one wagon, add 18.5 ft to tractor length. Wagon length is measured from front of drawbar to rear of wagon, and drawbar is 6.5 ft long.

**DESIGN-VEHICLE DIMENSIONS**

**Figure 40-4A**
BASIC DIMENSIONS OF DESIGN VEHICLE  
(Combination Truck A)  

Figure 40-4B
BASIC DIMENSIONS OF DESIGN VEHICLE
(Combination Truck B)

Figure 40-4C
<table>
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Note: Though this figure provides guidelines for determining FHWA oversight for each project, there may be exceptions, such as a project including a unique feature or an unusual circumstance such as a special structure design, experimental feature, etc., for which it is desirable to have FHWA oversight.

OVERSIGHT RESPONSIBILITY

Figure 40-7A
The MTMCTEA Design Exception Request Letter has been revised to the Surface Deployment and Distribution Command (SDDCTEA) Interstate Vertical Clearance Exception Coordination form. The form is available for download at http://www.in.gov/dot/div/contracts/design/dmforms/, Application/Request 40-8A
This figure deleted

The Level One Design Criteria Checklist has been renamed the Level One Controlling Criteria Checklist. The form and instructions for use are available for download at http://www.in.gov/dot/div/contracts/design/dmforms/, Checklist 40-8B

Editable Level One Design Criteria Checklist

Figure 40-8B
This figure deleted

The Level One Design Exception Request has been updated to reflect organizational changes in the Department. The editable form is available for download at http://www.in.gov/dot/div/contracts/design/dmforms/, Cover Letter/Memorandum 40-8C

Level One Design Exception Request
Figure 40-8C
CHAPTER 41

Highway Capacity

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.
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CHAPTER 41

HIGHWAY CAPACITY

Special Report 209 *Highway Capacity Manual (HCM)* has been adopted by the Department as the basic document for traffic-capacity analysis. This Chapter provides a supplement to the *HCM*. The supplement provides the following:

1. additional information on capacity-analysis procedures not provided in the *HCM*;
2. an elaboration on specific sections of the *HCM*;
3. clarifying information;
4. modifications to the *HCM* where the Department has adopted a different practice; and
5. the Department’s adopted practice, where the *HCM* indicates more than one option.

Unless stated otherwise in this Chapter, the Department has fully adopted the *HCM*. This Chapter is organized to follow the sequence shown in the *HCM*.

41-1.0 GENERAL

The *HCM* is the primary reference used to perform the Department’s traffic-capacity analysis. Another major source of information on capacity analysis is AASHTO’s *A Policy on Geometric Design of Highways and Streets*. Other sources on capacity analysis may be appropriate. However, prior to their use, the designer should first consult with the Production Management Division’s Office of Environmental Services and the Highway Operations Division’s Office of Traffic Engineering to confirm that these methodologies are applicable or acceptable to the Department.

Most of the methodologies included in the *HCM* are provided on a computerized software package entitled *Highway Capacity Software (HCS)*. The *HCS* package and the User’s Manual can be purchased from McTrans Center, 512 Weil Hall, Gainesville, Florida 32611-2083. The user should contact the Office of Environmental Services to determine which version may be used for capacity analysis. Other software packages which are based on the *HCM* may also be used, only after prior approval by INDOT. This approval will ensure that the software is an acceptable alternative to the *HCS*.
41-2.0 PRINCIPLES OF CAPACITY (HCM CHAPTERS 1 AND 2)

The following comment refers to Chapter 2, Traffic Characteristics.

Peak-Hour Factor. Existing traffic data should be used to determine the appropriate peak-hour factor. If the peak-hour factor cannot be determined from the existing traffic data, a peak-hour factor of 0.90 may be used. A factor as low as 0.60 may be used where significant peaking is expected to occur such as at a factory, industrial park, school, etc.

41-3.0 FREEWAY (HCM CHAPTERS 3, 4, 5, AND 6)

The following comments refer to Chapter 3, Basic Freeway Segments.

1. Truck-Lane Usage. Trucks are required to use the right lane of each roadway on a 4-lane freeway, or the right two lanes of a freeway of 6 lanes or more. Unless specific counts or observations are available, the truck distribution for a facility of 6 lanes or more can be assumed to be split evenly between the middle and right lanes. These configurations should be considered in making the freeway-capacity-analysis calculations.

2. Heavy-Vehicle Factor. Table 3-9, Adjustment Factor for the Effect of Trucks, Buses, or Recreational Vehicles in the Traffic Stream, should only be used if the traffic stream consists only of trucks, buses, or recreational vehicles, and not a combination of these vehicles. If the traffic stream consists of a combination of these vehicles, Equation 3-4 and the accompanying tables should be used instead.

41-4.0 RURAL HIGHWAY (HCM CHAPTERS 7 AND 8)

The following comment refers to Chapter 8, Two-Lane Highways.

Climbing Lane. Chapter 44 discusses the warrants for a climbing lane. The warrants described in Chapter 44 are different than those described in the HCM and AASHTO’s A Policy on Geometric Design of Highways and Streets.

41-5.0 URBAN STREET (HCM CHAPTERS 9, 10, 11, 12, 13, AND 14)

The following comments refer to Part IV, Urban Streets.
1. **Urban LOS.** For urban-highway elements, especially at a signalized intersection, a LOS of C may be difficult to attain. Often, a LOS of D is more attainable for a 10- to 20-year design. Chapters 53 and 55 provide the design LOS values.

2. **3-Lane Section.** The *HCM* does not directly address capacity of a continuous, alternating, or 2-way center left-turn lane. A 3-lane section with many left-turn movements will often have more capacity and greater safety than a 4-lane section without a separate turn lane. National studies are presently being conducted which may provide capacity information on these configurations in the near future.

The following comments refer to Chapter 9, Signalized Intersections.

1. **Planning Methodology.** Due to possible misapplications, the Planning Analysis should not be used for capacity analysis at a signalized intersection. Instead, the Operational Analysis procedure should be used.

2. **Operational Analysis.** After using the *HCM* procedure, the user should check this information using one of the signal-timing programs that are available (e.g., Passer II, SOAP84). See Section 502-3.0 for additional information.

3. **Level of Service (LOS).** As a guide, a lane-group LOS or approach LOS should not be more than one LOS below the intersection LOS or design LOS. However, this may not always be practical, especially for a left-turn lane or a group of side streets. Chapters 53 and 55 provide the LOS values required for design.

4. **Cycle Length.** The cycle length should be at least 60 s, but should not exceed 120 s. The degree of saturation should not be permitted to approach 1.0, especially for a short cycle length.
NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.
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CHAPTER 42

SIGHT DISTANCE

42-1.0 STOPPING SIGHT DISTANCE

42-1.01 Theoretical Discussion

Stopping sight distance (SSD) is the sum of the distance traveled during a driver’s perception/reaction or brake reaction time and the distance traveled while braking to a stop. To calculate SSD, the following formula is used:

\[ SSD = 1.47Vt + \frac{1.075V^2}{a} \]  

(Equation 42-1.1)

Where:
- \( SSD \) = stopping sight distance, ft
- \( V \) = design speed, mph
- \( t \) = brake reaction time, 2.5 s
- \( a \) = deceleration rate, 11.2 ft/s\(^2\)

The following discusses the theoretical rationale for each assumption within the SSD model.

1. **Brake Reaction Time.** This is the time interval between when an obstacle in the road can first be physically seen and when the driver first applies the brakes. The assumed value is 2.5 s. This time is considered adequate for 90% of drivers in simple to moderately-complex highway environments.

2. **Speed.** The SSD tables provide a minimum value which is based on the design speed.

3. **Grade Adjustment.** AASHTO’s *A Policy on Geometric Design of Highways and Streets* provides values to adjust the SSD for each grade which, theoretically, affects braking distances. Due to the conservative SSD model and the nature of the State’s terrain, the use of the grade adjustment is not required.

4. **AASHTO.** AASHTO’s *A Policy on Geometric Design of Highways and Streets* provides additional information on the assumptions used to develop the SSD model.
42-1.02 Passenger-Car Stopping Sight Distance

See Figure 42-1A, Stopping Sight Distance for Passenger Car. Stopping sight distance exceeding that shown in Figure 42-1A should be used where practical. In applying the SSD value for a passenger car, the height of eye is assumed to be 3.5 ft and the height of object 2 ft. The height of object is equivalent to the height of a passenger car’s taillights.

The minimum SSD value for a passenger car represents the Department’s Level One criterion for determining the need for a design exception. See Section 40-8.02.

42-1.03 Truck Stopping Sight Distance

Recommended stopping sight distance is based on passenger-car operation and does not explicitly consider design for truck operation. A truck, as a whole, especially a larger or heavier unit, needs a longer stopping sight distance for a given speed than does a passenger vehicle. However, the truck driver is able to see substantially farther beyond vertical sight obstructions because of the higher position of the seat in the vehicle. Separate stopping sight distance values for a truck and a passenger car are therefore not used in highway design.

Where horizontal sight restrictions occur on a downgrade, particularly at the end of a long downgrade where truck speed closely approaches or exceeds that of a passenger car, the greater height of a truck driver’s eye is of little value, even where the horizontal sight obstruction is a cut slope. Although the average truck driver tends to be more experienced than the average passenger-car driver and is quicker to recognize potential risks, it is desirable under such conditions to provide a stopping sight distance that exceeds the value shown in Figure 42-1A, Stopping Sight Distances for Passenger Cars.

42-2.0 DECISION SIGHT DISTANCE

42-2.01 Theoretical Discussion

A driver may be required to make a decision where the highway environment is difficult to perceive or where unexpected maneuvers are required. This occurs in an area of concentrated demand where the roadway elements, traffic volume, and traffic-control devices may all compete for the driver’s attention. This relatively complex environment may increase the required driver reaction time beyond that provided by the SSD value (2.5 s). At such a location, the designer should consider providing decision sight distance to provide an additional margin of safety. Decision sight distance reaction time ranges from 3 to 14.5 s depending on the location and
expected maneuver. The avoidance maneuvers used to develop Figure 42-2A, Decision Sight Distance, Columns A through E, are as follows:

2. Column B, Avoidance Maneuver B: Stop on urban road.
3. Column C, Avoidance Maneuver C: Speed/path/direction change on rural road.
5. Column E, Avoidance Maneuver E: Speed/path/direction change on urban road.

Columns A and B were developed using Section 42-1.0, Equation 42-1.1. Columns C, D, and E were developed using Equation 42-2.1, as follows:

\[
DSD = 1.47 \cdot Vt \quad \text{ (Equation 42-2.1)}
\]

where:
- \(DSD\) = decision sight distance, ft
- \(V\) = design speed, mph
- \(t\) = total time for the maneuver (reaction time + maneuver time), s

**42-2.02 Applications**

The designer should consider using decision sight distance at a relatively complex location where the driver reaction time may exceed 2.5 s. Example locations where decision sight distance may be appropriate include the following:

1. exit or entrance gore;
2. lane drop;
3. freeway left-side entrance or exit;
4. railroad/highway grade crossing;
5. approach to detour or lane closure;
6. toll plaza; or
7. intersection location where unusual or unexpected maneuvers are required.

As with SSD, the height of eye is 3.5 ft and the height of object is 2 ft.
42-3.0 PASSING SIGHT DISTANCE

42-3.01 Theoretical Discussion

Passing sight distance consideration is limited to a 2-lane, 2-way highway. On such a facility, a vehicle may overtake a slower-moving vehicle, and the passing maneuver must be accomplished on a lane used by opposing traffic.

The minimum passing sight distance is determined from the sum of four distances as illustrated in Figure 42-3A, Elements of Passing Sight Distance on a 2-Lane Highway. Figure 42-3B, Passing Sight Distance on a Two-Lane Highway, and the following provide the assumptions used to develop passing sight distance values.

1. Initial Maneuver Distance \( d_1 \). This is the distance traversed during the perception and reaction time and during the initial acceleration to the point of encroachment on the left lane. For the initial maneuver, the overtaken vehicle is assumed to be traveling at a uniform speed, and the passing vehicle is accelerating at the rate shown in Figure 42-3B. The average speed of the passing vehicle is assumed to be 10 mph higher than that of the overtaken vehicle. Equation 42-3.1 is used to determine \( d_1 \) as follows:

\[
d_1 = \frac{t_1}{0.68} \left( v - m + \frac{a t_1}{2} \right)
\]  

Where:
- \( t_1 \) = time of initial maneuver, s
- \( a \) = average acceleration, mph/s
- \( v \) = average speed of passing vehicle, mph
- \( m \) = difference in speed of passed vehicle and passing vehicle, mph

2. Distance that Passing Vehicle is in Left Lane \( d_2 \). This is the distance traveled by the passing vehicle while it occupies the left lane. The assumed time for while the passing vehicle occupies the left lane are shown in Figure 42-3B. Equation 42-3.2 is used to determine \( d_2 \) as follows:

\[
d_2 = \frac{v t_2}{0.68}
\]  

Where:
- \( t_2 \) = time during which the passing vehicle occupies the left lane, s
- \( v \) = average speed of passing vehicle, mph
3. **Clearance Distance** ($d_3$). This is the distance between the passing vehicle at the end of its maneuver and the opposing vehicle. This distance at the end of the passing maneuver is assumed to be between 100 ft and 250 ft.

4. **Opposing-Vehicle Distance** ($d_4$). This is the distance traversed by an opposing vehicle during two thirds of the time that the passing vehicle occupies the left lane. As shown in Figure 42-3A, the opposing vehicle appears after approximately one third of the passing maneuver ($d_2$) has been accomplished. The opposing vehicle is assumed to be traveling at the same speed as the passing vehicle. Therefore, $d_4 = \frac{2}{3} d_2$.

### 42-3.02 Applications

Figure 42-3B provides the minimum passing sight distance for design on a 2-lane, 2-way highway. This distance allows the passing vehicle to safely complete the passing maneuver. The value should not be confused with the value shown in the MUTCD for the placement of no-passing-zone stripes, which are based on different operational assumptions (i.e., distance for the passing vehicle to abort the passing maneuver). The highway capacity adjustment in the *Highway Capacity Manual* for a 2-lane, 2-way highway is based on the MUTCD criteria for marking a no-passing zone. It is not based on the percent of passing sight distance from AASHTO’s *A Policy on Geometric Design of Highways and Streets*.

For an existing highway, it will not be cost effective to improve the existing passing sight distance. On a rural new-construction or reconstruction project, the designer should attempt to provide passing sight distance over the project of the project consistent with the percentages shown in Figure 42-3C, Recommended Guideline For Percent Passing on Rural Facility. It will not be cost effective, however, to make significant improvements to the horizontal or vertical alignment solely to increase the available passing sight distance.

An appreciable grade can increase the sight distance required for safe passing. Passing tends to be easier for a vehicle traveling downgrade because the overtaking vehicle can accelerate more rapidly. However, so can the overtaken vehicle. For an upgrade, the passing sight distance should be greater than the derived minimum. Specific adjustments for use are unavailable. Consequently, the designer should use engineering judgment to make practical adjustments to the passing sight distance for an upgrade.

Passing sight distance is measured from a 3.5-ft height of eye to a 3.5-ft height of object. It is impractical to design a crest vertical curve to provide for passing sight distance because of high cost where a cut are involved.
42-4.0 INTERSECTION SIGHT DISTANCE

Section 46-10.0 discusses the design requirements for intersection sight distance.
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**STOPPING SIGHT DISTANCE FOR PASSENGER CAR**

*Figure 42-1A*
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
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<th>Avoidance Maneuver B</th>
<th>Avoidance Maneuver C</th>
<th>Avoidance Maneuver D</th>
<th>Avoidance Maneuver E</th>
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<td>535</td>
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<td>780</td>
<td>1410</td>
<td>1105</td>
<td>1275</td>
<td>1445</td>
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Avoidance Maneuver A: Stop on rural road – \( t = 3.0 \) s
Avoidance Maneuver B: Stop on urban road – \( t = 9.1 \) s
Avoidance Maneuver C: Speed/path/direction change on rural road – \( t \) varies between 10.2 and \( 11.2 \) s
Avoidance Maneuver D: Speed/path/direction change on suburban road – \( t \) varies between 12.1 and \( 12.9 \) s
Avoidance Maneuver E: Speed/path/direction change on urban road – \( t \) varies between 14.0 and \( 14.5 \) s

**DECISION SIGHT DISTANCE**

**Figure 42-2A**

*Note: Figures 42-2B, 42-2C, 42-2D, and 42-2E have been deleted.*
ELEMENTS OF PASSING DISTANCE
(2-Lane Highways)

Figure 42-3A
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Assumed Speeds</th>
<th>Passing Sight Distance</th>
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<tr>
<td></td>
<td>Passed Vehicle (mph)</td>
<td>Passing Vehicle (mph)</td>
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<tr>
<td>20</td>
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<td>60</td>
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PASSING SIGHT DISTANCE ON TWO-LANE HIGHWAY

Figure 42-3B
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<tr>
<th>Terrain</th>
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<td>Arterial</td>
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<td>Level</td>
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<td>Rolling</td>
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**RECOMMENDED GUIDELINE FOR PERCENT PASSING**  
*(Rural)*  

**Figure 42-3C**
NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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<td>May 2013</td>
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CHAPTER 43

HORIZONTAL ALIGNMENT

43-1.0 DEFINITIONS

The definitions for basic elements of horizontal alignment are shown below. Section 43-6.0 provides mathematical details for a horizontal curve (e.g., deflection angle, point of curvature).

1. Simple Curve. This is a continuous arc of constant radius which achieves the necessary highway deflection without an entering or exiting transition.

2. Compound Curves. This is a series of two or more simple curves with deflections in the same direction immediately adjacent to each other.

3. Reverse Curves. This is two simple curves with deflections in opposite directions which are joined by a relatively short tangent distance.

4. Broken-Back Curves. This is two closely-spaced horizontal curves with deflection angles in the same direction with an intervening, short tangent section.

5. Superelevation. Superelevation is the amount of cross slope or banking provided on a horizontal curve to help counterbalance the centrifugal force of a vehicle traversing the curve.

6. Maximum Superelevation, $e_{\text{max}}$. This is an overall superelevation control used on a specific facility. Its selection depends on factors including climatic conditions, terrain conditions, type of area (rural or urban), and highway functional classification.

7. Superelevation Transition Length. This is the distance required to transition the roadway from a normal crown section to full superelevation. It is the sum of the tangent runout, $TR$, and superelevation runoff, $L$, distances, as follows:

   a. Tangent Runout, $TR$. This is the distance required to change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed (i.e., the outside lane is level).
b. **Superelevation Runoff, \( L \).** This is the distance required to change the cross slope from the end of the tangent runout (adverse cross slope removed) to a section that is sloped at the design superelevation rate.

8. **Axis of Rotation.** This is the line about which the pavement is revolved to superelevate the roadway. This line will maintain the normal highway profile throughout the curve.

9. **Superelevation Rollover.** This is the algebraic difference, \( A \), between the superelevated travel-lane slope and shoulder slope on the outside of a horizontal curve.

10. **Normal Crown (NC).** This is the typical cross section on a tangent section (i.e., no superelevation).

11. **Remove Adverse Crown (RC).** This is a superelevated roadway section which is sloped across the entire traveled way in the same direction and at a rate equal to the cross slope on a tangent section.

12. **Relative Longitudinal Slope.** In a superelevation-transition portion of a two-lane facility, this is the relative gradient between the profile grade and edge of traveled way.

### 43-2.0 HORIZONTAL CURVE

#### 43-2.01 General Theory

A horizontal curve is, in effect, a transition between two tangents. These deflectional changes are necessary in virtually all highway alignments to avoid impacts on a variety of field conditions (e.g., right-of-way, natural features, man-made features). The following provides a brief overview of the general theory of horizontal alignment. The designer should reference the AASHTO *A Policy on Geometric Design of Highways and Streets* for more information.

#### 43-2.01(01) Basic Curve Equation

The point-mass formula is used to define vehicular operation around a curve. Where the curve is expressed using its radius, the basic equation for a simple curve is as follows:

\[
R = \frac{V^2}{15(e + f)}
\]
Where:

\[
\begin{align*}
R & = \text{radius of curve, ft} \\
e & = \text{superelevation rate} \\
f & = \text{side-friction factor} \\
V & = \text{vehicular speed, mph}
\end{align*}
\]

43-2.01(02) Theoretical Approaches

Establishing horizontal-curvature criteria requires a determination of the theoretical basis for the various factors in the basic curvature equation. These include the side-friction factor, \( f \), and the distribution method between side friction and superelevation. The theoretical basis will be one of the following.

1. **Open-Roadway Condition.** The theoretical basis includes the following:
   a. relatively low side-friction factor (i.e., a relatively small level of driver discomfort); and
   b. the use of AASHTO Method 5 to distribute side friction and superelevation.

   Open-roadway condition applies to a rural facility or an urban facility where the design speed \( V \geq 50 \text{ mph} \).

2. **Low-Speed Urban Street.** The theoretical basis includes the following:
   a. relatively high side-friction factor to reflect a higher level of driver acceptance of discomfort; and
   b. the use of AASHTO Method 2 to distribute side friction and superelevation.

   A low-speed urban street is defined as that within an urban or urbanized area where the design speed \( V \leq 45 \text{ mph} \).

3. **Turning-Roadway Condition.** The theoretical basis includes the following:
   a. higher side-friction factor than open-roadway condition to reflect a higher level of driver acceptance of discomfort; and
b. a range of acceptable superelevation rates for combinations of curve radius and design speed to reflect the need for flexibility to meet field conditions for a turning roadway.

This applies to a turning roadway at an intersections at-grade. See Chapter Forty-six.

43-2.01(03) Superelevation

Superelevation allows a driver to negotiate a curve at a higher speed than would otherwise be comfortable. Superelevation and side friction work together to offset the outward pull of the vehicle as it traverses the horizontal curve. It is necessary to establish a limiting value of superelevation rate, $e_{max}$, based on the operational characteristics of the facility. Values of $e_{max}$ used by INDOT are discussed in Section 43-3.0.

43-2.01(04) Side Friction

AASHTO has established limiting side-friction factors, $f$, for various design speeds and various highway operating conditions. The $f$ value represents a threshold of driver discomfort, and not the point of impending skid. Different sets of $f$ values have been established for different operating conditions (i.e., open roadway, low-speed urban street, or turning roadway). The basis for the distinction is that drivers, through conditioning, will accept a different level of discomfort on each different facility.

43-2.02 Selection of Horizontal-Curve Type

Because of its simplicity and ease of design, survey, and construction, a simple curve is nearly always used on the highway mainline. A simple curve may rarely be inconsistent with field conditions; therefore, an alternative arrangement such as a compound curve should be used. Spiral curves should not be used.

43-2.03 Minimum Radius

The following figures provide the minimum radius, $R_{min}$, for an open-roadway facility or a low-speed urban street. Criteria for a turning roadway are provided in Chapter Forty-six. To define $R_{min}$, a maximum superelevation rate, $e_{max}$, must be selected. These are as follows:
1. Figure 43-2A is applicable to a facility where $e_{\text{max}} = 8\%$ and open-roadway conditions apply.

2. Figure 43-2B is applicable to a low-speed urban street where $e_{\text{max}} = 4\%$ or $6\%$ is applied.

See Section 43-3.0 for the selection of $e_{\text{max}}$ for various facility types.

43-2.04 Maximum Deflection Without Curve

It may be appropriate to design a facility without a horizontal curve where small a deflection angle is present. As a guide, the designer may retain a deflection angle of about 1 deg or less (urban), or 0.5 deg or less (rural) for the highway mainline. The absence of a horizontal curve will not likely affect driver response or aesthetics.

43-2.05 Minimum Length of Curve

A short horizontal curve may provide the driver the appearance of a kink in the alignment. To improve the aesthetics of the highway, the designer should lengthen each short curve, if practical, even if not necessary for engineering reasons. The following guidance should be used to compare the calculated curve length to the recommended minimum length.

1. General. The minimum length of curve on an open roadway should be based on the deflection angle, $\Delta$, as follows:

<table>
<thead>
<tr>
<th>$\Delta$ (deg)</th>
<th>Minimum Curve Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 1$</td>
<td>100</td>
</tr>
<tr>
<td>$1 &lt; \Delta \leq 2$</td>
<td>200</td>
</tr>
<tr>
<td>$2 &lt; \Delta \leq 3$</td>
<td>300</td>
</tr>
<tr>
<td>$3 &lt; \Delta \leq 4$</td>
<td>400</td>
</tr>
<tr>
<td>$4 &lt; \Delta \leq 5$</td>
<td>500</td>
</tr>
<tr>
<td>$&gt; 5$</td>
<td>Calculated Length</td>
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</tbody>
</table>

The minimum length of curve on a low-speed urban street will be determined as required.

2. Freeway or Rural Highway. The minimum length of curve in feet should be $15V$ for aesthetics. $V$ is design speed in mph.
**43-2.06 Shoulder Treatment**

On a facility with a relatively sharp horizontal curve, the calculated and design values for traveled-way-widening on an open-highway curve (two-lane highway, one-way, or two-way), shown in the AASHTO *A Policy on Geometric Design of Highways and Streets*, and truck volume greater than 1000, a full-structural strength shoulder should be provided on both sides of the curve in lieu of pavement widening. The following will apply.

1. **Strengthened Length.** The strengthened shoulder should be available from the beginning of the superelevation transition before the curve to the end of the transition beyond the curve.

2. **Asphalt Traveled Way.** The pavement structure of the strengthened shoulder should match that of the traveled way.

3. **Concrete Traveled Way with Asphalt Shoulder.** The Office of Pavement Engineering will determine the pavement structure of the strengthened shoulder.

4. **Concrete Traveled Way with Concrete Shoulder.** The pavement structure of the strengthened shoulder should match that of the traveled way.

See AASHTO *A Policy on Geometric Design of Highways and Streets* for more information on pavement widening.

**43-3.0 SUPERELEVATION**

**43-3.01 Superelevation Rate, Open-Roadway Condition**

**43-3.01(01) General**

The open-roadway condition is used for each rural highway, or each urban facility where $V \geq 50$ mph. This type of facility exhibits relatively uniform traffic operations. Therefore, for superelevation development, the flexibility exists to design a horizontal curve with the more conservative AASHTO Method 5 (for distribution of superelevation and side friction). The following provides the specific design criteria for superelevation rate assuming the open-roadway condition.
43-3.01(02) Maximum Superelevation Rate

The selection of a maximum rate of superelevation, $e_{max}$, depends upon urban or rural location and prevalent climatic conditions. For the open-roadway condition, INDOT has adopted the following for the selection of $e_{max}$.

1. **Rural Facility.** An $e_{max} = 8\%$ is used. An exception should be evaluated as required.

2. **Urban Facility ($V \geq 50$ mph).** An $e_{max} = 8\%$ is used. A rate of 6\% or 4\% may be used where $V \leq 45$ mph or where site-specific conditions warrant.

43-3.01(03) Superelevation Rate

Based on the selection of $e_{max} = 8\%, 6\%, \text{ or } 4\%$ and the use of AASHTO Method 5 to distribute $e$ and $f$, Figures 43-3A(1), 43-3A(2), and 43-3A(3) allow the designer to select the superelevation rate for any combination of curve radius, $R$, and design speed, $V$. The design speed selected for determining the superelevation rate will be the same as that used for the overall project design. However, site-specific factors may indicate a need to use a higher design speed specifically to determine the superelevation rate. This may be appropriate if the designer anticipates that travel speeds higher than the project design speed will occur at the horizontal curve with some frequency. Examples include the following.

1. **Transition Area.** Where a highway is transitioning from a predominantly rural environment to an urban environment, travel speeds in the transition area within the urban environment may be higher than the urban design speed.

2. **Downgrade.** Where a horizontal curve is located at the bottom of a downgrade, travel speeds at the curve may be higher than the overall project design speed. As suggested adjustments, the design speed used for the horizontal curve may be 5 mph (grade of 3\% to 5\%) or 10 mph (grade >5\%) higher than the project design speed. This adjustment may be more appropriate for a divided facility than for a 2-lane, 2-way highway.

3. **Long Tangent.** Where a horizontal curve is located at the end of a long tangent section, a design speed of up to 10 mph higher than the project design speed may be appropriate.

43-3.01(04) Minimum Radius Without Superelevation
A horizontal curve with a very large radius does not require superelevation, and the normal crown section (NC) used on the tangent section can be maintained throughout the curve. On a sharper curve for the same design speed, a point is reached where a superelevation rate of 2% across the total traveled way width is appropriate. Figure 43-3B provides the threshold (or minimum) radius for a normal crown section at various design speeds. The figure also provides the curve-radius range where remove (adverse) crown (RC) applies. This table applies to where the open-roadway condition is used.

43-3.02 Superelevation Rate, Low-Speed Urban Street

43-3.02(01) General

In a built-up area, the combination of wide pavements, proximity of adjacent development, control of cross slope, profile for drainage, frequency of cross streets, and other urban features make superelevation impractical and undesirable. Superelevation is not provided on a local street in a residential area. It may be considered on a local street in an industrial area to facilitate operation. If superelevation is used, the curve should be designed for a maximum superelevation rate of 4%. If terrain dictates sharp curvature, a maximum superelevation rate of 6% is justified if the curve is long enough to provide an adequate superelevation transition.

The low-speed urban street condition may be used for a superelevating street in an urban or urbanized area where \( V \leq 45 \) mph. A superelevation rate of 6% is considered the maximum desirable rate for low-speed urban street design. On such a facility, providing superelevation at a horizontal curve is frequently impractical because of roadside conditions and may result in undesirable operational conditions. The following lists some of the characteristics of a low-speed urban street which often complicate superelevation development.

1. **Roadside Development, Intersection, or Drive.** Built-up roadside development is commonly adjacent to a low-speed urban street. Matching a superelevated curve with a drive, intersection, sidewalk, etc., creates considerable complications. This may also require re-shaping a parking lot, lawn, etc., to compensate for the higher elevation of the high side of the superelevated curve.

2. **Non-Uniform Travel Speed.** Travel speeds are often non-uniform because of frequent signalization, stop signs, vehicular conflicts, etc. It is undesirable for traffic to stop on a superelevated curve, especially if snow or ice is present.

3. **Limited Right of Way.** Superelevating a curve often results in more right-of-way impacts than would otherwise be necessary. Right of way is often restricted.
4. **Wide Pavement Area.** A low-speed urban street may have wide pavement areas because of high traffic volume in a built-up area, the absence of a median, or the presence of parking lanes. The wider the pavement area, the more complicated will be the development of superelevation.

5. **Surface Drainage.** Proper pavement drainage can be difficult with a normal crown. Superelevation introduces another complicating factor.

As discussed in Section 43-2.0, AASHTO Method 2 is used to distribute superelevation and side friction in determining the superelevation rate for the design of a horizontal curve on a low-speed urban street. A relatively high side-friction factor is used. The practical impact is that superelevation is rarely warranted on such a facility.

The higher side-friction factor for a low-speed urban street is consistent with driver acceptance of more discomfort in an urban area.

### 43-3.02(02) Superelevation Rate

Figure 43-3C is used to determine the superelevation rate for a horizontal curve of given radius on a low-speed urban street of given design speed. The figure is divided into three areas. The following examples illustrate how to use Figure 43-3C for site conditions within each area.

* * * * * * * *

**Example 43-3.1**

Given: 
- Design speed = 35 mph
- Radius = 600 ft
- Cross slope (on tangent) = 2%

Problem: Determine the superelevation rate.

Solution: From Figure 43-3C the required superelevation rate = -0.043. Since this value is negative, a normal crown section should be maintained throughout the curve (i.e., e = -0.020).
**Example 43-3.2**

Given: Design speed = 35 mph  
Radius = 450 ft  

Problem: Determine the superelevation rate.

Solution: From Figure 43-3C, the required superelevation rate = +0.006. This occurs in the area where the roadway may be uniformly superelevated at the cross slope of the roadway on tangent (typically 0.020). This is the desirable treatment. However, it is acceptable to superelevate the roadway at the theoretical superelevation rate (+0.006), if this is consistent with field conditions (e.g., surface drainage will work properly).

**Example 43-3.3**

Given: Design speed = 35 mph  
Radius = 390 ft  

Problem: Determine the superelevation rate.

Solution: Figure 43-3C yields a required superelevation rate = +0.03. Therefore, the entire pavement should be transitioned to this rate.

* * * * * * * * *

**43-3.02(03) Minimum Radius Without Superelevation**

On a low-speed urban street, a horizontal curve with a sufficiently large radius does not require superelevation; therefore, the normal crown section can be maintained around a curve. The threshold exists where the theoretical superelevation equals -0.02. The lower boundary of the shaded area in Figure 43-3C illustrates this threshold. For convenience, see Figure 43-3D, Curve Radius for Normal-Crown Section and Remove (Adverse)-Crown Section (Low-Speed Urban Street).
43-3.03 Transition Length, Open-Roadway Condition

As defined in Section 43-1.0, the superelevation transition length is the distance required to transition the roadway from a normal crown section to the full design superelevation (as determined from the figures based on the selected $e_{\text{max}}$). The superelevation transition length is the sum of the tangent runout distance, $TR$, and superelevation runoff length, $L$.

43-3.03(01) Two-Lane Roadway

1. **Superelevation Runoff.** Figure 43-3A(1) shows the superelevation runoff length, $L_2$, for various combinations of curve radius and design speed. The length is calculated as follows:

   $$L_2 = W e(RS)$$

   (Equation 43-3.1)

   Where:

   $L_2$ = Superelevation runoff length (assuming the axis of rotation is about the roadway centerline), ft
   $W$ = Width of rotation (assumed to be 12 ft)
   $e$ = Superelevation rate
   $RS$ = Reciprocal of relative longitudinal slope between the profile grade and outside edge of roadway (see Figure 43-3E)

   The superelevation runoff length applies to the following:

   a. a 2-lane, 2-way roadway rotated about its centerline; or
   
   b. either directional roadway of a 4-lane divided facility, rotated about its centerline independently of the other roadway [see Section 43-3.03(02)].
2. **Tangent Runout.** The tangent runout distance is calculated as follows:

\[
TR = \frac{L_2 S_{normal}}{e}
\]

(Equation 43-3.2)

Where:

\(TR\) = Tangent runout distance, ft

\(L_2\) = Superelevation runoff length, ft (Equation 43-3.1)

\(S_{normal}\) = Travel lane cross slope on tangent (typically 0.02)

\(e\) = Design superelevation rate (i.e., full superelevation for horizontal curve)

This will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

**43-3.03(02) Highway with 4 or More Lanes**

1. **Superelevation Runoff.** The superelevation runoff distance is calculated as follows:

\[
L = \frac{wn_t e b_w}{G}
\]

(Equation 43-3.3)

Where:

\(L\) = Superelevation runoff length, ft, rounded up to the next 15-ft increment

\(w\) = Width of one traffic lane, ft

\(n_t\) = Number of lanes rotated

\(e\) = Design superelevation rate, %

\(b_w\) = Adjustment factor for number of lanes rotated (see Figure 43-3G)

\(G\) = Maximum relative gradient, %
2. **Tangent Runout.** The tangent runout distance is calculated from Equation 43-3.2, same as for a two-lane roadway.

The length of tangent runout is determined by the amount of adverse cross slope to be removed and the rate at which it is removed. To effect a smooth edge of pavement profile, the rate of removal should equal the relative gradient used to define the superelevation runoff length.

The cross slope may not be constant across all lanes. If there are three lanes sloped in the same direction, the first two lanes will be sloped at 2% and the third will be sloped at 3%. See Section 45-1.01(02) Item 2.b.

This will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

### 43-3.03(03) Application of Transition Length

Once the superelevation runoff and tangent runout superelevation transition length have been calculated, the designer must determine how to fit the length in the horizontal and vertical planes. The following will apply:

1. **Simple Curve.** Typically, 75% of the superelevation runoff length will be placed on the tangent and the remainder on the curve. Exceptions to this practice may be necessary to meet field conditions. The superelevation runoff may be distributed 50% to 70% on the tangent and 50% to 30% on the curve. It is acceptable to use Figure 43-3F to determine the percent of superelevation runoff to place on the tangent before the PC.

2. **Reverse Curve.** See Section 43-3.07 for a discussion on superelevation development for a reverse curve.

3. **Vertical Profile.** At the beginning and ending of the superelevation transition, angular breaks would occur in the profile if it is not smoothed. These abrupt angular breaks should be smoothed by the insertion of short vertical curves at the two angle points. As a guide, the transitions should have a length of 60 ft.

4. **Ultimate Development.** If the facility is planned for ultimate development to an expanded facility, the designer should, where practical, reflect this in the initial superelevation-transition application. For example, a four-lane divided facility may be planned to ultimately be a six-lane divided facility. Therefore, the superelevation runoff
length for the initial four-lane facility should be consistent with the future requirements of the six-lane facility. See Section 43-3.05.

43-3.03(04) Superelevation-Development Figures

Figures 43-3H, 43-3I, 43-3J, and 43-3K are the figures for superelevation development. The following describes each figure.

1. Two-Lane Roadway. Figure 43-3H illustrates the superelevation development for a 2-lane roadway. The axis of rotation is about the centerline of the roadway.

2. Four-Lane Divided with No Future Third Lane. Figure 43-3I illustrates the superelevation development for this situation. The axes of rotation are about the two median edges.

3. Six-Lane Divided or Four-Lane Divided with Future Third Lane. Figure 43-3J illustrates the superelevation development for this situation. The axes of rotation are about the two median edges or, where the future third lane is anticipated in the median, about the two future median edges. The figure illustrates how to treat the travel lane with a steeper cross slope (i.e., 3%).

4. Median Barrier. Figure 43-3K illustrates the superelevation development for a divided highway with a median barrier. The axes of rotation are about the two edges of the median barrier, which allows the barrier to remain within a horizontal plane throughout the horizontal curve. The figure illustrates how to treat the two inside shoulders in the superelevation development.

These figures provide acceptable methods for superelevation development which will often be applicable to typical site conditions. Other superelevation methods or strategies should be developed as required to meet specific field conditions. For example, several highway features may significantly influence superelevation development for a divided highway. These include guardrail, median barrier, drainage, or other field conditions. The designer should consider the intended functions of these features and ensure that the superelevated section and selected axis of rotation does not compromise their operation. The acceptability of superelevation-development methods other than those in the figures should be judged individually.

For a divided facility, the figures provide the superelevation development for the inside and outside roadways separately. The coordination between the two roadways for a given station number will be determined individually. The superelevation development for each roadway
should begin such that full superelevation for each roadway is reached simultaneously (i.e., at the same station).

43-3.04 Transition Length, Low-Speed Urban Street

A low-speed urban street is an urban facility where \( V \leq 45 \) mph. If the open-roadway condition is used to determine the superelevation rate, the superelevation transition length should be determined by means of the criteria for the open-roadway condition (Section 43-3.03). If the superelevation rate is determined by means of the low-speed urban street condition, the superelevation transition length may be determined by means of the criteria described below.

43-3.04(01) Two-Lane Roadway

1. **Superelevation Runoff.** Figure 43-3L provides the minimum superelevation runoff length, \( L_2 \), for a 2-lane roadway. Using a straight-line interpolation to determine an intermediate superelevation rate, the superelevation runoff may be calculated for any design speed and superelevation rate.

   If \( L_2 \) is less than the value of \( L_r \) shown in AASHTO *Policy on Geometric Design of Highways and Streets* Exhibit 3-32, use the value shown in the Exhibit.

   For a site-specific situation, the Exhibit 3-32 value of \( L_r \) may not be attainable. If so, a Level One design exception request should be submitted for approval.

2. **Tangent Runout.** The tangent runout distance can be calculated from Equation 43-3.2, using \( L_2 \) from Figure 43-3L. This will ensure that the relative longitudinal gradient of the tangent runout equals that of the superelevation runoff.

43-3.04(02) Highway with 4 or More Lanes

Section 43-3.03 provides criteria for superelevation transition length for such a highway assuming the open-roadway condition. This is accomplished by providing an adjustment factor, \( C \), to apply to the transition length, \( L_2 \), for a 2-lane, 2-way roadway. The procedures and formulas in Section 43-3.03 also apply to a highway with 4 or more lanes assuming the low-speed urban street condition, except that \( L_2 \) will be based on Figure 43-3L.
43-3.04(03) Application of Transition Length

The criterion provided in Section 43-3.03 for the open-roadway condition also applies to a low-speed urban street.

43-3.05 Axis of Rotation

The following discusses the axis of rotation for a 2-lane, 2-way highway or highway with 4 or more lanes. Section 43-3.03 provides figures illustrating the application of the axis of rotation in superelevation development.

43-3.05(01) Two-Lane, Two-Way Highway

The axis of rotation will be about the centerline of the roadway. This method will yield the least amount of elevation differential between the pavement edges and their normal profiles. It is acceptable to rotate about the inside or outside edge of the travelway. This may be necessary to meet field conditions (e.g., drainage on a curbed facility, roadside development).

On a 2-lane highway with an auxiliary lane (e.g., a climbing lane), the axis of rotation will be about the centerline of the two through lanes.

43-3.05(02) Divided Highway

If no future travel lanes are planned, the axes of rotation will be about the two median edges. Where these are used as the axes, the median will remain in a horizontal plane throughout the curve. Depending upon field conditions, the axes of rotation may be about the centerlines of the two roadways. Unless the two roadways are on independent alignments, this method results in different elevations at the median edges and, therefore, a compensating slope is necessary across the median. On a narrow median, the axis of rotation may be about the centerline of the entire roadway cross section.

The figures in Section 43-3.03 illustrate the axis of rotation for a divided highway.
43-3.06 Shoulder Superelevation

43-3.06(01) High-Side Shoulder

The following will apply to the shoulder slope.

1. **Application.** The high-side shoulder will be sloped as follows:

   a. If the superelevation rate on the curve is 4% or less, use 4% (its normal cross slope).

   b. If the superelevation rate on the curve is greater than 4% but less than or equal to 6%, use 2% down away from the traveled way.

   c. If the superelevation rate on the curve is greater than 6%, use 1% towards the traveled way.

   d. Where the paved median shoulder is the high-side shoulder and is 4 ft or narrower, it should be sloped in the same plane as the travelway. See Figure 43-3M, Paved-Shoulder Cross Slopes, Superelevated Section, With Underdrains; or Figure 43-3N, Paved-Shoulder Cross Slopes, Superelevated Section, Without Underdrains, for more-specific information.

2. **Maximum Rollover.** Where the typical application cannot be provided, the high-side shoulder must be sloped such that the algebraic difference between the shoulder and adjacent travel lane will not exceed 8%.

3. **Shoulder as Deceleration Lane.** A driver may use a paved shoulder as a right-turn lane on a superelevated horizontal curve. Chapter Forty-six provides cross-slope breakover criteria between a turning roadway and a through travel lane at an intersection at-grade. Where the shoulder is used by a turning vehicle, the designer should limit the shoulder rollover to the turning roadway breakover criteria (4% to 5%).

43-3.06(02) Low-Side (Inside) Shoulder

The normal shoulder slope should be retained until the adjacent superelevated travel lane reaches that slope. The shoulder is then superelevated concurrently with the travel lane until the design superelevation is reached (i.e., the inside shoulder and travel lane will remain in a plane section).
43-3.07 Reverse Curve

A reverse curve is two closely-spaced simple curves with deflections in opposite directions. For this situation, it may not be practical to achieve a normal crown section between the curves. A plane section continuously rotating about its axis (e.g., the centerline) can be used between the two curves, if they are close enough together. The applicable superelevation-development criteria should be used for each curve. The following will apply to a reverse curve.

1. **Normal Section.** The designer should not attempt to achieve a normal tangent section between the two curves unless the normal section can be maintained for a minimum of two seconds of travel time, and the superelevation-transition requirements can be met for both curves.

2. **Continuously-Rotating Plane.** If a normal section is not provided, the pavement will be continuously rotated in a plane about its axis. The minimum distance between the PT and PC will be that needed to meet the superelevation-transition requirements for the two curves (e.g., distribution of superelevation runoff between the tangent and curve).

43-3.08 Bridge

If practical, a horizontal curve or superelevation transition should be avoided on a bridge. A bridge should be placed within a curve if this results in a more desirable alignment on either approaching roadway. If a superelevation transition is unavoidable on a bridge, see Section 59-1.01(01) for recommendations. However, if properly designed and constructed, a bridge will function adequately where this occurs.

43-4.0 HORIZONTAL SIGHT DISTANCE

43-4.01 Sight Obstruction Definition

A sight obstruction on the inside of a horizontal curve is defined as an obstacle of considerable length which continuously interferes with the line of sight. This includes a guardrail, bridge railing, median barrier, wall, cut slope, wooded area, building, or tall farm crop. A barrier to the line of sight should be assumed to be constructed on the right-of-way line. A point obstacle such as a traffic sign or utility pole is not considered a sight obstruction. The designer must examine each curve individually to determine whether it is necessary to remove an obstruction, increase
the offset to the obstruction, or increase the radius to obtain the required sight distance. However, the shoulder width should not exceed 12 ft.

43-4.02 Curve Length Relative to Stopping Sight Distance

1. Curve Length > Stopping Sight Distance. Where the length of curve, $L$, is greater than the stopping sight distance, $S$, used for design, the needed clearance on the inside of the horizontal curve is calculated as follows:

$$ M = R \left[ 1 - \cos \left( \frac{28.65S}{R} \right) \right] $$

(Equation 43-4.1)

Where:

$ M = $ Middle ordinate, or distance from the center of the inside travel lane to the obstruction, ft

$ R = $ Radius of curve, ft

$ S = $ Stopping sight distance, ft

2. Curve Length $\leq$ Stopping Sight Distance. Where the length of curve is less than or equal to the stopping sight distance, the design should be checked graphically or by utilizing a computational method.

43-4.02(01) Stopping Sight Distance (SSD)

At a minimum, SSD will be available throughout the horizontal curve. Figure 43-4A provides the horizontal clearance criteria (i.e., middle ordinate) for various combinations of stopping sight distance and curve radius. For those selections of $S$ which appear outside of the range of values in the figure (i.e., $M > 50$ ft or $R < 165$ ft), the designer should use Equation 43-4.1 to calculate the needed clearance. The Example in Figure 43-4C illustrates the determination of clearance requirements for entering or exiting from a horizontal curve.
43-4.02(02) Other Sight Distance Criteria

It may be warranted to provide SSD for trucks, or decision sight distance or passing sight distance at the horizontal curve. Chapter Forty-two discusses candidate sites and provides design values for such sight-distance criteria. These \( S \) values should be used in the basic equation to calculate \( M \) (Equation 43-4.1).

43-4.02(03) Entering and Exiting Portions

The \( M \) value from Figure 43-4A applies between the PC and PT. Some transition is needed on the entering and exiting portions of the curve. The procedure is as follows.

1. Locate the point which is on the outside edge of shoulder and a distance of \( S/2 \) before the PC.

2. Locate the point which is a distance \( M \) measured laterally from the center of the inside travel lane at the PC.

3. Connect the two points located in Steps 1 and 2. The area between this line and the roadway should be clear of all continuous obstructions.

4. A symmetrical application of Steps 1 through 3 should be used beyond the PT.

The Example in Figure 43-4C illustrates the determination of clearance requirements for entering or exiting from a curve.

43-4.03 Application

For application, the height of eye is 3.5 ft and the height of object is 2 ft. Both the eye and object are assumed to be in the center of the inside travel lane. If the lane width for a ramp is wider than 12 ft, the horizontal stopping sight distance should be calculated by placing the eye and object 6 ft from the edge of the lane on the inside of the curve.
43-4.04 Longitudinal Barrier

A longitudinal barrier (e.g., bridge railing, guardrail, median barrier) can cause sight distance problems at a horizontal curve, since a barrier is placed relatively close to the travel lane (often, 10 ft or less) and its height is greater than 2 ft.

The designer should check the line of sight over a barrier along a horizontal curve and attempt, if practical, to locate the barrier such that it does not block the line of sight. The following should be considered.

1. **Superelevation.** The designer should account for the superelevation in the calculations.

2. **Grade.** The line of sight over a barrier may be improved for a driver on an upgrade or lessened on a downgrade.

3. **Barrier Height.** The higher the barrier, the more obstructive it will be to the line of sight.

Each barrier location on a horizontal curve will require an individual analysis to determine its impacts on the line of sight. The designer must determine the height of the driver’s eye, the height of the object, and the height of the barrier where the line of sight intercepts the barrier run. If the barrier does block the line of sight to a 2-ft height object, the designer should consider relocating the barrier or revising the horizontal alignment. If the barrier blocks the sight distance needed for minimum SSD on the mainline, it will be necessary to obtain a design exception.

43-5.0 DESIGN CONTROLS AND PROCEDURE

43-5.01 General Controls

As discussed in Chapter Forty-three, the design of horizontal alignment involves complying with specific limiting criteria. These include minimum radius, superelevation rate, and sight distance around a curve. Certain design principles and controls should be considered which will determine the overall safety of the facility and will enhance the aesthetic appearance of the highway. These design principles include the following.

1. **Consistency.** Alignment should be consistent. A sharp curve at the end of a long tangent, or a sudden change from gently- to sharply-curving alignment should be avoided.

2. **Direction.** Alignment should be as directional as possible, consistent with physical and economic constraints. On a divided highway, a flowing line that conforms generally to
the natural contours is preferable to one with long tangents that slash through the terrain. Directional alignment will be achieved by using the smallest practical central angle.

3. **Use of Minimum Radius.** The use of the minimum radius should be avoided if practical.

4. **High Fill.** Avoid a sharp curve on a long, high fill. Under this condition, it is difficult for a driver to perceive the extent of horizontal curvature.

5. **Alignment Reversal.** Avoid an abrupt reversal in alignment, such as an S or reverse curve. Provide a sufficient tangent distance between two curves to ensure proper superelevation transitions for both curves.

6. **Broken-Back Curvature.** Avoid this where possible. This arrangement is not aesthetically pleasing, it violates driver expectancy, and it creates undesirable superelevation-development requirements.

7. **Compound Curve.** Avoid the use of a compound curve on the highway mainline. This may fool the driver when judging the sharpness of a horizontal curve.

8. **Coordination with Natural or Man-Made Feature.** The horizontal alignment should be properly coordinated with the existing alignment at the ends of the project, natural topography, available right-of-way, utilities, roadside development, or natural or man-made drainage patterns.

9. **Environmental Impact.** Horizontal alignment should be properly coordinated with environmental impact (e.g., encroachment onto wetlands).

10. **Intersection.** Horizontal alignment through an intersection may present problems (e.g., intersection sight distance, superelevation development). See Chapter Forty-six for the design of an intersection at-grade.

11. **Coordination with Vertical Alignment.** Chapter Forty-four discusses design principles for the coordination between horizontal and vertical alignments.

**43-5.02 Coordination**

In the design of horizontal alignment, the designer should be aware of the responsibility to communicate properly with other INDOT personnel (e.g., drafting, field survey):
1. **Preparation of Plans.** Part II discusses the content and format of plans sheets, abbreviations, symbols, scales, and the use of the Department's CADD system. The designer must ensure that the design of the horizontal alignment is consistent with Department practices.

2. **Surveying.** Part III provides the Department's procedures and criteria for surveying practice.

3. **Mathematical Computations.** Section 43-6.0 provides figures which include the needed mathematical equations and techniques to make various computations for a horizontal curve.

**43-6.0 MATHEMATICAL DETAILS FOR HORIZONTAL CURVE**

This Section provides mathematical details used for various applications to the design of a horizontal curve. Figure 43-6A summarizes the figures in the Section.
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\[
R_{min} = \frac{V^2}{15(e + f_{max})} \quad \text{where} \quad e = 0.08
\]

Note: The value of $R_{min}$ for design has been rounded to the nearer 5-ft increment.

MINIMUM RADIUS
Open-Roadway Conditions

Figure 43-2A
<table>
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<tr>
<th>Design Speed, $V$ (mph)</th>
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<th>$f_{\text{max}}$</th>
<th>Minimum Radius, $R_{\text{min}}$ (ft)</th>
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$$R_{\text{min}} = \frac{V^2}{15(e + f_{\text{max}})}$$

*Note: The value of $R_{\text{min}}$ for design has been rounded up to the nearer 5-ft increment*

**MINIMUM RADIUS**

Low-Speed Urban Street, $V \leq 45$ mph

*Figure 43-2B*
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**Note:** Use of $e_{max} = 4\%$ should be limited to urban conditions.

- $e = 1.5$ is Normal Crown.
- $e = 2.0$ is Remove (Adverse) Crown

**MINIMUM RADIUS, $R$, FOR DESIGN SUPERELEVATION RATE, $e$, DESIGN SPEED, $V_d$, AND $e_{max} = 4\%$**

*Figure 43-3A(1)*
$e = 1.5$ is Normal Crown.

$e = 2.0$ is Remove (Adverse) Crown

**MINIMUM RADIUS, \( R \), FOR DESIGN SUPERELEVATION RATE, \( e \), DESIGN SPEED, \( V_d \), AND \( e_{\text{max}} = 6\% \)**

**Figure 43-3A(2)**

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$e = 1.5$ is Normal Crown.
$e = 2.0$ is Remove (Adverse) Crown

**MINIMUM RADIUS, R, FOR DESIGN SUPERELEVATION RATE, e,**
**DESIGN SPEED, $V_d$, AND $\epsilon_{\text{max}} = 8\%$**

Figure 43-3A(3)
Normal Crown and Remove (Adverse) Crown curve radii can be found on Figures 43-3A(1), 43-3A(2), and 43-3A(3).

CURVE RADII FOR NORMAL CROWN AND REMOVE CROWN SECTIONS

Figure 43-3B
Notes:

1. Figure denotes three areas for the determination of superelevation rates. See Section 43-3.02 for examples on how to use the figure.

2. The basic equation for the figure is: 
   \[ R = \frac{V^2}{15(e + f)} \]

   Where:
   - \( R \) = curve radius, ft.
   - \( V \) = design speed, mph
   - \( e \) = super elevation rate
   - \( f \) = side-friction factor

3. Negative superelevation values beyond -2.0 percent should be used for a low-type surface such as gravel, crushed stone, or earth. However, a normal cross slope of -2.5 percent can be used on a high-type surface in an area with intense rainfall.

SUPERELEVATION RATE FOR
LOW-SPEED URBAN STREET

Figure 43-3C
<table>
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<th>Design Speed, $V$ (mph)</th>
<th>Curve Radius, $R$ (ft)</th>
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* The shaded area in Figure 43-4C reflects these radius ranges. In one of these ranges, it is desirable to remove the crown and superelevate the roadway at a uniform cross slope, $e$, of +0.02. However, it is acceptable to superelevate at the theoretical rate from Figure 43-3C, if consistent with field conditions.

Note: The limit for normal crown is based on a theoretical superelevation rate, $e$, of -0.02. The upper limit for remove (adverse) crown is based on a theoretical superelevation rate, $e$, of +0.02. The radius is calculated from the formula as follows:

$$ R = \frac{V^2}{15(e + f)} $$

**RADIUS FOR NORMAL-CROWN SECTION AND REMOVE (ADVERSE)-CROWN SECTION (Low-Speed Urban Street)**

Figure 43-3D
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Equivalent Max. RS</th>
<th>Edge-of-Travelway Slope Relative to Centerline $G_{max}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>128</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>135</td>
<td>0.74</td>
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<tr>
<td>25</td>
<td>143</td>
<td>0.70</td>
</tr>
<tr>
<td>30</td>
<td>152</td>
<td>0.66</td>
</tr>
<tr>
<td>35</td>
<td>161</td>
<td>0.62</td>
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<tr>
<td>40</td>
<td>172</td>
<td>0.58</td>
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<td>45</td>
<td>185</td>
<td>0.54</td>
</tr>
<tr>
<td>50</td>
<td>200</td>
<td>0.50</td>
</tr>
<tr>
<td>55</td>
<td>213</td>
<td>0.47</td>
</tr>
<tr>
<td>60</td>
<td>222</td>
<td>0.45</td>
</tr>
<tr>
<td>65</td>
<td>233</td>
<td>0.43</td>
</tr>
<tr>
<td>70</td>
<td>250</td>
<td>0.40</td>
</tr>
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$G_{max} = \frac{100}{RS}$

**RELATIVE LONGITUDINAL SLOPES**

*(Two-Lane Roadway)*

**Figure 43-3E**
<table>
<thead>
<tr>
<th>V (mph)</th>
<th>Number of Lanes Rotated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>15 - 45</td>
<td>80%</td>
</tr>
<tr>
<td>50 - 70</td>
<td>70%</td>
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</tbody>
</table>

PORTION OF SUPERELEVATION RUNOFF ON TANGENT, %

Figure 43-3F
<table>
<thead>
<tr>
<th>Number of Lanes Being Rotated*</th>
<th>$b_w$</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
</tr>
<tr>
<td>1½</td>
<td>0.83</td>
</tr>
<tr>
<td>2</td>
<td>0.75</td>
</tr>
<tr>
<td>2½</td>
<td>0.70</td>
</tr>
<tr>
<td>3</td>
<td>0.67</td>
</tr>
<tr>
<td>3½</td>
<td>0.64</td>
</tr>
</tbody>
</table>

* This column refers to the number of lanes being rotated on either side of the axis rotation. Select the higher value.

As an example, consider a 5-lane roadway (i.e., four through lanes and a two-way, left-turn lane (TWLTL) with the axis of rotation in the center of the TWLTL. In this case, the number of lanes being rotated is 2.5; therefore, $b_w = 0.70$.

bw VALUES
(Superelevation Runoff Lengths, Multilane Highways)

Figure 43-3G
SUPERELEVATION DEVELOPMENT
(Two-Lane Roadways)

Figure 43-3H
OUTSIDE ROADWAY

NOTES:
1. AXES OF ROTATION ABOUT TWO MEDIAN EDGES
2. e = APPLICABLE SUPERELEVATION RATE

SUPERELEVATION DEVELOPMENT
(Four-Lane Divided, No Future Third Lane)

Figure 43-3 I
INSIDE ROADWAY

NOTES:  
1. AXES OF ROTATION ABOUT TWO MEDIAN EDGES  
2. e = APPLICABLE SUPERELEVATION RATE

SUPERELEVATION DEVELOPMENT  
(Four-Lane Divided, No Future Third Lane)
OUTSIDE ROADWAY

NOTES:
1. AXES OF ROTATION ABOUT TWO MEDIAN EDGES
2. e = APPLICABLE SUPERELEVATION RATE

SUPERELEVATION DEVELOPMENT
(Six-Lane (or more) Divided)
(Four-Lane Divided with Future Additional Lanes)

Figure 43-3J
INSIDE ROADWAY

SUPERELEVATION DEVELOPMENT
(Six-Lane (or more) Divided)
(Four-Lane Divided with Future Additional Lanes)

Figure 43-3J
(Continued)
SUPERELEVATION DEVELOPMENT
(With Concrete Median Barrier)

Figure 43-3K
(Page 1 of 2)
NOTES:
1. AXES OF ROTATION ABOUT EDGES OF CMB
2. $e =$ APPLICABLE SUPERELEVATION RATE

SUPERELEVATION DEVELOPMENT
(with Concrete Median Barrier)

Figure 43-3K
(Page 2 of 2)
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Superelevation Rate, ( e )</th>
<th>Minimum Superelevation Runoff, ( L_2 ) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0.02</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>0.03</td>
<td>51</td>
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<td></td>
<td>0.04</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td>0.05</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td>0.06</td>
<td>103</td>
</tr>
<tr>
<td>30</td>
<td>0.02</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>0.03</td>
<td>54</td>
</tr>
<tr>
<td></td>
<td>0.04</td>
<td>73</td>
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<tr>
<td></td>
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<tr>
<td>35</td>
<td>0.02</td>
<td>39</td>
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<td></td>
<td>0.03</td>
<td>58</td>
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<tr>
<td></td>
<td>0.04</td>
<td>77</td>
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<tr>
<td>40</td>
<td>0.02</td>
<td>41</td>
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<td></td>
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<td>62</td>
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<td>83</td>
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<td>0.06</td>
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<td></td>
<td>0.03</td>
<td>66</td>
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<tr>
<td></td>
<td>0.04</td>
<td>89</td>
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<tr>
<td></td>
<td>0.05</td>
<td>111</td>
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<tr>
<td></td>
<td>0.06</td>
<td>133</td>
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</tbody>
</table>

Note: For a superelevation rate intermediate between those in table, use a straightline interpolation to calculate the superelevation runoff length.

SUPERELEVATION RUNOFF LENGTH  
(Low-Speed Two-Lane Urban Street)

Figure 43-3L
<table>
<thead>
<tr>
<th>Paved Shld. Width, ( w ) (ft)</th>
<th>High-Side-Shoulder Cross Slope</th>
<th>Low-Side-Shoulder Cross Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 2 \leq w \leq 4 )</td>
<td>( e )</td>
<td>( e )</td>
</tr>
<tr>
<td>( w &gt; 4 )</td>
<td>( e ) for 2 ft Closest to Travel Lane, then **</td>
<td>( e ) for 2 ft Closest to Travel Lane, then ***</td>
</tr>
</tbody>
</table>

\( e \) = superelevation rate for travelway

** as outlined in Section 43-3.06(01)

*** as outlined in Section 43-3.06(02)

**PAVED-SHOULDER CROSS SLOPES
SUPERELEVATED SECTION, WITH UNDERDRAINS**

Figure 43-3M
Paved Shld. Width, \( w \) (ft) & High-Side-Shoulder Cross Slope & Low-Side-Shoulder Cross Slope \\
\hline
0 \leq w \leq 2 & \( e \) & \( e \) \\
2 < w \leq 4 & \( e \) & \( e \) \\
\( w > 4 \) & ** & *** \\
\hline

\( e \) = superelevation rate for travelway \\
** as outlined in Section 43-3.06(01) \\
*** as outlined in Section 43-3.06(02)

**PAVED-SHOULDER CROSS SLOPES**
**SUPERELEVATED SECTION, WITHOUT UNDERDRAINS**

**Figure 43-3N**
DESIGN CONTROLS FOR STOPPING SIGHT DISTANCE ON HORIZONTAL CURVE

Figure 43-4A
KEY:

\[ H = \text{Horizontal Sight Line Offset (ft)} \]
\[ S = \text{Stopping Sight Distance (ft)} \]
\[ R = \text{Radius of Curve (ft)} \]

EXAMPLE:

Given: \( \text{Design Speed} = 55 \text{ mph}, R = 1000 \text{ ft} \)

Problem: Determine the horizontal clearance requirements for the horizontal curve.

Solution: Use the equation for horizontal clearance \( (L > S) \) to obtain

\[
H = R \left[ 1 - \cos \left( \frac{28.65 \cdot S}{R} \right) \right]
\]

\[
H = 1000 \left[ 1 - \cos \left( \frac{28.65 \cdot 495}{1000} \right) \right] = 30.5 \text{ ft}
\]

NOTE: This figure also illustrates the horizontal clearance requirements for the entering and exiting portion of the horizontal curve.

HORIZONTAL SIGHT DISTANCE CLEARANCE REQUIREMENTS

Figure 43-4C
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<td>Simple Curve Computation</td>
</tr>
<tr>
<td>Figure 43-6D</td>
<td>Curve Symbols, Abbreviations and Formulas</td>
</tr>
<tr>
<td>Figure 43-6E</td>
<td>Simple Curve Computation (Example)</td>
</tr>
<tr>
<td>Figure 43-6F</td>
<td>Simple Curves (Stationing)</td>
</tr>
</tbody>
</table>

MATHEMATICAL DETAILS FOR HORIZONTAL CURVES

Figure 43-6A
Let BC = a, AC = b, AB = c. Then:

1. \( \sin \alpha = \frac{a}{c} \)

2. \( \cos \alpha = \frac{b}{c} \)

3. \( \tan \alpha = \frac{a}{b} \)

4. \( \csc \alpha = \frac{1}{\sin \alpha} = \frac{c}{a} \)

5. \( \sec \alpha = \frac{1}{\cos \alpha} = \frac{c}{b} \)

6. \( \cot \alpha = \frac{1}{\tan \alpha} = \frac{b}{a} \)

7. \( \text{vers} \alpha = 1 - \cos \alpha \)

8. \( \text{covers} \alpha = 1 - \sin \alpha \)

9. \( \text{exsec} \alpha = (\sec \alpha) - 1 \)

10. \( \text{coexsec} \alpha = (\csc \alpha) - 1 \)

11. \( a^2 + b^2 = c^2 \)

12. \( \alpha + \beta = 90^\circ \)

13. \( \text{Area} = \frac{1}{2} ab \)

**BASIC TRIGONOMETRIC FUNCTIONS**

*Figure 43-6B*
CONTROL-POINT ABBREVIATIONS

PC = Point of Curvature (beginning of curve)

PI = Point of Intersection of tangents

PT = Point of Tangency (end of curve)

PRC = Point of Reverse Curvature

PCC = Point of Compound Curvature

FORMULAS

\[ L = \frac{\Delta R \pi}{180} \]

\[ T = R \tan \left( \frac{\Delta}{2} \right) \]

\[ E = T \tan \left( \frac{\Delta}{4} \right) = \left[ \frac{R}{\cos \left( \frac{\Delta}{2} \right)} \right] - R \]

\[ LC = 2R \sin \left( \frac{\Delta}{2} \right) \]

\[ M = R \left[ 1 - \cos \left( \frac{\Delta}{2} \right) \right] = E \cos \left( \frac{\Delta}{2} \right) \]

SYMBOLS

\[ \Delta = \text{Deflection angle (deg)} \]

\[ T = \text{Tangent length (distance from PC to PI, or from PI to PT) (ft)} \]

\[ L = \text{Length of curve (distance from PC to PT along curve) (ft)} \]

\[ R = \text{Radius of curve (ft)} \]

\[ E = \text{External distance (transverse distance from PI to midpoint of curve) (ft)} \]

\[ LC = \text{Long Chord length (straight-line distance from PC to PT) (ft)} \]

\[ C = \text{midpoint of long Chord} \]

\[ M = \text{Middle ordinate distance (transverse distance from midpoint of L to point C (ft)} \]

LOCATING THE PC OR PT

Station of PC = Station of PI – T/100

Station of PT = Sta. of PC + L/100

1 station = 100 ft. For example, Sta. 13+54.86 is 1354.86 ft from Sta. 0+00.00.

HORIZONTAL CURVE ABBREVIATIONS, SYMBOLS, AND FORMULAS

Figure 43-6D
Sample Problem:

With the alignment information given below, determine the basic curve data.

Solution:

From the information given, find L and T:

\[ L = PT \text{ Sta.} - PC \text{ Sta.} = (20 + 77.72) - (16 + 64.78) = 412.94 \text{ ft} \]
\[ T = PI \text{ Sta.} - PC \text{ Sta.} = (18 + 55.36) - (16 + 64.78) = 190.58 \text{ ft} \]

Using horizontal curve formulas from Figure 43-6D, solve for E, M, and R:

\[ R = \frac{T}{\tan(\frac{\Delta}{2})} = \frac{19.058}{\tan 19.17^\circ} = \frac{19.058}{0.34765} = 54.820 \text{ ft} \]
\[ E = T \tan(\frac{\Delta}{4}) = (190.58)(\tan 9.585E) = (190.58)(0.16887) = 32.18 \text{ ft} \]
\[ M = R (1 - \cos \frac{\Delta}{2}) = (412.94)(1 - \cos 19.17E) = (412.94)(0.05545) = 22.90 \text{ ft} \]

SIMPLE CURVE COMPUTATION

(Example)

Figure 43-6E
1. The station at the first PI is 6+18.54.
2. The station at the first PC = 618.54 + 224.05 = 3+94.49.
3. The station at the first PT = 394.49 + 438.71 = 8+33.20.
4. The station at the second PC = 833.20 + (838.98 + 224.05 + 247.87) = 12+00.26.
5. The station at the second PI = 1200.26 + 247.87 = 14+48.13.
6. The station at the second PT = 1200.26 + 479.42 = 16+79.68.
7. The station at the third PC = 1679.68 + 939.07 + 247.87 = 261.45 = 21+09.43.
8. The station at the third PI = 2109.43 + 261.45 = 23+70.88.
9. The station at the third PT = 2109.54 + 500.20 = 26+09.63.
10. The station at the final POT = 2609.63 + 678.08 = 678.08 = 30+26.26.
11. Check: (618.54 + 838.98 + 939.07 + 678.08) - (2 x 224.05 + 2 x 247.87 + 2 x 261.45 - 438.71 479.42 500.20) = 3026.26.

**SIMPLE CURVES STATIONING**

**Figure 43-6F**
CHAPTER 44

Vertical Alignment

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.
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<td>K-Values for Crest Vertical Curves (Decision Sight Distance - Passenger Cars)</td>
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CHAPTER 44

VERTICAL ALIGNMENT

This Chapter provides the Department’s criteria for the design of each vertical-alignment element. This includes grade, climbing lane, vertical curve, and vertical clearance.

44-1.0 GRADE

44-1.01 Terrain Definitions

1. Level. Highway sight distances are either long or could be made long without major construction expense. The terrain is considered to be flat, which has minimal impact on vehicular performance.

2. Rolling. The natural slopes consistently rise above and fall below the roadway grade. Steep slopes may restrict the desirable highway alignment. Rolling terrain generates steeper grades, causing trucks to reduce speeds to below those of passenger cars.

3. Mountainous. Longitudinal and transverse changes in elevation are abrupt, and benching and side-hill excavation are frequently required to provide the desirable highway alignment. Mountainous terrain aggravates the performance of trucks relative to passenger cars, resulting in some trucks operating at crawl speeds.

The use of mountainous terrain criteria will not be permitted on a Federal-aid project because, even though a roadway may pass through a mountainous site, the area as a whole is still considered to be rolling terrain.

If it is not clear which terrain designation to use (e.g., level versus rolling), the flatter of the two should be selected.

44-1.02 Maximum Grade

Chapters 53 through 56 provide the Department’s criteria for maximum grade based on functional classification, urban or rural location, type of terrain, design speed, and project scope of work. The maximum grade should be used only where absolutely necessary. Where practical, a grade flatter than the maximum should be used.
44-1.03 Minimum Grade

The following provides the Department’s criteria for minimum grade.

1. **Uncurbed Road.** It is desirable to provide a longitudinal grade of approximately 0.5%. This allows for the possibility that the original crown slope is subsequently altered as a result of swell, consolidation, maintenance operations, or resurfacing. A level longitudinal grade may be acceptable on a pavement which is adequately crowned to drain laterally.

2. **Curbed Street.** The centerline profile on a highway or a street with curbs should desirably have a minimum longitudinal grade of 0.5%. A flatter or level grade with rolling curb lines may be necessary in level terrain, where the adjacent development precludes the taking of additional right of way.

On a curbed facility, the longitudinal grade at the gutter line will have a significant impact on the pavement drainage characteristics (e.g., ponding, flow capture by grated inlets or catch basins). See Part IV for more information on pavement drainage.

44-1.04 Critical Length of Grade

Critical length of grade is the maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. The highway gradient in combination with the length of grade will determine the truck speed reduction on an upgrade. The following will apply to the critical length of grade.

1. **Design Vehicle.** A loaded truck, powered so that the mass/power ratio is about 200 lb/hp is representative of the size and type of vehicle normally used for design on a major route. For another type of highway, designing for the 200 lb/hp truck is not always cost-effective, especially on a route which has minimal truck traffic. Therefore, to better reflect the wide range of trucks, INDOT has adopted the following critical-length-of-grade criteria.

   a. **Major Route.** The 10-mph reduction curve shown in Figure 44-1A, Critical Length of Grade for Truck, provides the critical length of grade for a 200 lb/hp truck. This figure should be used to determine the critical length of grade on a freeway, principal or minor arterial, or for a project on the extra-heavy-duty-highway system. See Chapter 60 for a listing of extra-heavy-duty routes. It also should be used on another type of road classification where significant numbers of large trucks are known to use the facility (e.g., coal-hauling route).
b. Other Route. The 15-mph reduction curve shown in Figure 44-1A provides the critical length of grade for a single-unit truck and the major portion of tractor-trailer trucks.

See Figure 44-1B, Critical Length of Grade for Recreational Vehicles.

2. Criteria. Figure 44-1A provides the critical lengths of grade for a given percent grade and acceptable truck-speed reduction. This figure is based on an initial truck speed of 70 mph, and representative truck of 200 lb/hp.

3. Momentum Grade. Where an upgrade is preceded by a downgrade, a truck will often increase speed to make the climb. A speed increase of 10 mph on a moderate downgrade (3 to 5%), and 15 mph on a steeper downgrade (6 to 8%) of sufficient length are reasonable adjustments. These can be used in design to allow the use of a higher speed reduction curve from Figure 44-1A or 44-1B. However, this speed increase may not be attainable if traffic volume is high enough that a truck may be behind a passenger vehicle when descending the momentum grade. Therefore, the increase in speed can only be considered if the highway has a LOS of C or better.

4. Measurement. Figures 44-1A and 44-1B are based upon length of tangent grade. If a vertical curve is part of the length of grade, Figure 44-1C, Measurement for Length of Grade, illustrates how to determine an approximate equivalent tangent grade length.

5. Application. If the critical length of grade is exceeded, the grade should be flattened, if practical, or the need for a truck-climbing lane should be evaluated (see Section 44-2.0).

6. Highway Type. The critical-length-of-grade criteria apply to a 2-lane or divided highway, or to an urban or rural facility. A climbing lane is not used as extensively on a freeway or multilane facility since it more frequently has sufficient capacity to handle its design-year traffic without being congested. A faster vehicle can more easily move left to pass a slower vehicle.

7. Example Problems. Examples 44-1.1 and 44-1.2 illustrate the use of Figure 44-1A to determine the critical length of grade. Example 44-1.3 illustrates the use of both Figures 44-1B and 44-1C. In the examples, the use of subscripts 1, 2, etc., indicate the successive grades and lengths of grade on the highway segment.

* * * * * * * * * * *
Example 44-1.1

Given:  Level Approach
\( G = +4\% \)
\( L = \text{Length of grade of 1000 ft} \)
Rural Arterial

Problem:  Determine if the critical length of grade is exceeded.

Solution:  Figure 44-1A yields a critical length of grade of 1150 ft for a 10-mph speed reduction.  The grade is therefore acceptable (1000 ft < 1150 ft).

Example 44-1.2

Given:  Level Approach
\( G_1 = +2\% \)
\( L_1 = 1600 \text{ ft} \)
\( G_2 = +5\% \)
\( L_2 = 650 \text{ ft} \)
Rural Collector with significant number of heavy trucks

Problem:  Determine if the critical length of grade is exceeded for the combination of grades \( G_1 \) and \( G_2 \)

Solution:  Using Figure 44-1A, \( G_1 \) yields a truck speed reduction of 5 mph.  \( G_2 \) yields approximately 6 mph.  The total of 11 mph is greater than the allowable 10 mph.  Therefore, the critical length of grade is exceeded.

Example 44-1.3

Given:  Figure 44-1D illustrates the vertical alignment on a low-volume, 2-lane rural highway with no large trucks.

Problem:  Determine if the critical length of grade is exceeded for \( G_2 \) or the combination upgrade \( G_3/G_4 \).
Solution: Figure 44-1C provides the criteria for determining the length of grade. This is calculated as follows for this example.

\[
L_2 = \frac{1000}{4} + 600 + \frac{850}{4} = 1062 \text{ ft}
\]

\[
L_3 = \frac{850}{4} + 650 + \frac{410}{2} = 1068 \text{ ft}
\]

\[
L_4 = \frac{410}{2} + 500 + \frac{790}{4} = 903 \text{ ft}
\]

Read into Figure 44-1B for \( G_2 \) (3%) and find a length of grade of 1800 ft. \( L_2 \) is less than this value, therefore the length of grade is not exceeded.

Read into Figure 44-1B for \( G_3 \) (3.5%) and \( L_3 = 1080 \) ft and find a speed reduction of 4 mph. Read into Figure 44-1B for \( G_4 \) (2%) and \( L_4 = 900 \) ft and find a speed reduction of 2 mph. Therefore, the total speed reduction on the combination upgrade \( G_3/G_4 \) is 6 mph. However, for a low-volume road, the designer may assume a 5-mph increase in truck speed for the 3% momentum grade, \( G_2 \), which precedes \( G_3 \). Therefore, the speed reduction may be as high as 15 mph before the combination grade exceeds the critical length of grade. Assuming the benefits of the momentum grade leads to the conclusion that the critical length of grade is not exceeded.

**********

44-2.0 CLIMBING LANE

44-2.01 Warrants

A climbing lane may be warranted for truck or recreational-vehicle traffic so that a specific upgrade can operate at an acceptable level of service. The following criteria will apply.

44-2.01(01) Two-Lane Highway

A climbing lane may be warranted if the following conditions are satisfied.

1. Upgrade traffic flow rate is in excess of 200 vehicles per hour.

2. Upgrade truck flow rate is in excess of 20 trucks per hour.
3. One of the following conditions exists.
   
   a. A 10-mph or greater speed reduction is expected for a typical heavy truck.
   
   b. Level of Service (LOS) of E or F exists on the grade.
   
   c. A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

The upgrade flow rate is determined by multiplying the design-hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak-hour factor. See AASHTO *A Policy on Geometric Design of Highways and Streets* for more information including where to begin and end a climbing lane.

A climbing lane may also be warranted where the above criteria are not met if, for example, there is an adverse accident experience on the upgrade related to slow-moving trucks. However, on a designated recreational route, where a low percentage of trucks may not warrant a climbing lane, sufficient recreational-vehicle traffic may indicate a need for an additional lane. This can be evaluated by using Figure 44-1B, Critical Length of Grade for Recreational Vehicle. A climbing lane must be designed for each traffic direction, independently of the other.

### 44-2.01(02) Divided Highway

A climbing lane may be warranted if the following conditions are satisfied.

1. The critical length of grade is less than the length of grade being evaluated; and

2. one of the following conditions exists:
   
   a. the LOS on the upgrade is E or F, or
   
   b. there is a reduction of one or more LOS when moving from the approach segment to the upgrade; and

3. the construction costs and the construction impacts (e.g., environmental, right of way) are considered reasonable.
A climbing lane is generally not warranted on a 4-lane facility with directional volume below 1000 vehicles per hour per lane, regardless of the percentage of trucks. See AASHTO *A Policy on Geometric Design of Highways and Streets* for more information.

A climbing lane may also be warranted where the above criteria are not met if, for example, there is an adverse accident experience on the upgrade related to slow-moving trucks.

**44-2.02 Capacity Procedure**

**44-2.02(01) Two-Lane Highway**

The objective of the capacity analysis procedure is to determine if the warranting criteria in Section 44-2.01 are met for a 2-lane facility. This is accomplished by calculating the service flow rate for each LOS level (A through D) and comparing this to the actual flow rate on the upgrade. Because a LOS worse than D warrants a climbing lane, it is not necessary to calculate the service flow rate for LOS of E.

The operations on the grade should be analyzed using the procedures in the *Highway Capacity Manual* (HCM). In addition, the following should be considered.

1. To calculate the LOS, the following data should be compiled to complete the analysis.
   a. Average annual daily traffic (AADT) (mixed composition for year under design);
   b. the $K$ factor (i.e., the proportion of AADT occurring in the design hour);
   c. the directional distribution, $D$, during the design hour (DHV);
   d. the truck factor, $T$, during the DHV (i.e., the percent of trucks, buses, and recreational vehicles);
   e. the peak-hour factor, $PHF$;
   f. the design speed;
   g. lane and shoulder width (ft);
   h. percent grade;
i. percent no-passing zones (based on the MUTCD criteria for striping of a no-passing zone); see Section 502-2.0; and

j. length of grade (mi).

2. The type of truck is not a factor in determining the passenger-car equivalent. Only the proportion of heavy vehicles (i.e., trucks, buses, or recreational vehicles) in the upgrade traffic stream is applicable.

3. For a highway with a single grade, the critical length of grade can be directly determined from Figure 44-1A, Critical Length of Grade for Truck, or Figure 44-1B, Critical Length of Grade for Recreational Vehicle. However, the highway will usually have a continuous series of grades. It is necessary to find the impact of a series of significant grades in succession. If several different grades are present, a speed profile may need to be developed. Section 44-2.04 provides information on how to develop a truck speed profile.

44-2.02(02) Divided Highway

A climbing lane on a divided highway is not as easily justified as that on a 2-lane facility because of the operational advantage of divided highway. A passenger car can pass a slow-moving truck without occupying an opposing lane of travel. As indicated in Section 44-2.01, INDOT has adopted criteria to warrant a truck-climbing lane on a divided highway. These are based on the critical length of grade and on the LOS on the upgrade.

The calculation of LOS for an upgrade is similar to that for a 2-lane highway; see Section 44-2.02(01) and the HCM. However, the adjustment factors required to calculate the service flow rate differ. This reflects the operational difference between a divided and a 2-lane facility. See the Highway Capacity Manual for the detailed capacity methodology.

44-2.03 Design

See Figure 44-2A, Design Criteria for Climbing Lane. The following should also be considered.

1. Design Speed. For a design speed of 55 mph or higher, use 55 mph for truck design speed. For a speed lower than 55 mph, use the design speed.

2. Superelevation. For a horizontal curve, the climbing lane will be superelevated at the same rate as the adjacent travel lane.
3. **Performance Curve.** Figure 44-2B, Performance Curves for Heavy Truck (200 lb/hp) for Deceleration on Upgrade, provides the deceleration rates for a heavy truck. Figure 44-2C, Speed-Distance Curves for Acceleration of a Typical Heavy Truck (200 lb/hp) on Upgrade or Downgrade, provides the acceleration rates for a heavy truck.

4. **End of Full-Width Lane.** In addition to the criteria in Figure 44-2A, the available sight distance should be considered to the point where the truck will merge back into the through travel lane. At a minimum, this will be stopping sight distance. The driver should have decision sight distance available to the merge point at the end of the taper to safely complete the maneuver, especially where the merge is on a horizontal or vertical curve.

**44-2.04 Truck-Speed Profile**

The following example illustrates how to construct a truck-speed profile and how to use Figures 44-2B and 44-2C.

* * * * * * * * * *

**Example 44-2.1**

Given: Level Approach  
$G_1 = +3\%$ for 500 ft (PVI to PVI)  
$G_2 = +5\%$ for 3500 ft (PVI to PVI)  
$G_3 = -2\%$ beyond the composite upgrade ($G_1$ and $G_2$)  
$V = 60$ mph (design speed)  
Rural Arterial, Heavy-Truck Route

Problem: Using the criteria shown in Figure 44-2A and Figure 44-2B, construct a truck-speed profile and determine the beginning and ending points of the full-width climbing lane.

Solution: The following steps apply.

Step 1: Determine the beginning of the full-width climbing lane. From Figure 44-2A, the beginning of the full-width lane will begin at the PVC and, at a minimum, at the PVT.

Step 2: Determine the truck speed on $G_i$, at 200-ft increments, using Figure 44-2B and plot them in Figure 44-2D. Assume an initial truck speed of 55 mph (see Figure 44-2B).
### Distance From PVI1 (ft) | Horizontal Distance on Figure 44-2B (ft) | Truck Speed (mph) | Comments
---|---|---|---
0 | 0 | 55 | PVI1
200 | 200 | 53 |
400 | 400 | 51 |
500 | 500 | 50 | PVI2

**Step 3:** Determine the truck speed on $G_2$, at 500-ft increments, using Figure 44-2B and plot them in Figure 44-2D. From Step 2, the initial speed on $G_2$ is the final speed from $G_1$ (i.e., 50 mph). Move left horizontally along the 50-mph line to the 5% upgrade. This is approximately 250 ft along the horizontal axis. This is the starting point for $G_2$.

### Distance From PVI1 (ft) | Horizontal Distance on Figure 44-2B (ft) | Truck Speed (mph) | Comments
---|---|---|---
500 | 1500 | 55 | PVI2
1000 | 2000 | 50 |
1500 | 2500 | 45 |
2000 | 3000 | 40 |
2500 | 3500 | 36 |
3000 | 4000 | 32 |
3500 | 4500 | 30 (1) |
4000 | 5000 | 27 (1) | PVI3

(1) The final crawl speed of the truck for a 5% upgrade.

**Step 4:** Determine the truck speed on $G_3$, at 500-ft increments, using Figure 44-2B until the point where the truck is able to accelerate to 45 mph (minimum design speed for ending the climbing lane) and plot them in Figure 44-2D. The truck will have a speed of 27 mph as it enters the 2% downgrade at the PVI3. Read into Figure 44-2B at the 27-mph point on the vertical axis over to the -2% line. This is approximately 0 ft along the horizontal axis. The -2% line is followed to 45 mph, which is approximately 1000 ft along the horizontal axis. Therefore, the truck will require 1000 ft (1000 ft - 0 ft) from the PVI3 to reach 45 mph. The truck will require approximately an additional 1200 ft to reach 55 mph (the desirable criterion).
Distance From PVI₁ (ft) | Horizontal Distance on Figure 44-2C (ft) | Truck Speed (mph) | Comments
--- | --- | --- | ---
4000 | 0 | 27 | PVI₃
4500 | 500 | 40 | 
5000 | 1000 | 45 | Minimum End
5500 | 1500 | 50 | 
6000 | 2000 | 53 | 
6500 | 2500 | 55 | Desirable End

********

### 44-3.0 VERTICAL CURVE

#### 44-3.01 Crest Vertical Curve

A crest vertical curve is in the shape of a parabola. The basic equations for determining the minimum length of a crest vertical curve are as described below.

#### 44-3.01(01) Stopping Sight Distance

If the stopping sight distance, $S$, is less than the vertical curve length, $L$,

$$ L = \frac{AS^2}{100 (\sqrt{2 h_1} + \sqrt{2 h_2})^2} = \frac{AS^2}{2158} \quad \text{(Equation 44-3.1)} $$

$$ L = KA \quad \text{(Equation 44-3.2)} $$

If the stopping sight distance, $S$, is greater than or equal to the vertical curve length, $L$,

$$ L = 2S - \frac{2158}{A} \quad \text{(Equation 44-3.3)} $$

where:

$L$ = length of vertical curve, ft

$A$ = algebraic difference between the two tangent grades, %
\[ S = \text{stopping sight distance, ft} \]
\[ h_1 = \text{height of eye above road surface, ft} \]
\[ h_2 = \text{height of object above road surface, ft} \]
\[ K = \text{horizontal distance needed to produce a 1\% change in gradient} \]

The length of the crest vertical curve will depend upon \( A \) for the specific curve and upon the selected sight distance, height of eye, and height of object. The following discusses the selection of these values.

The principal control in the design of a crest vertical curve is to ensure that, at a minimum, stopping sight distance (SSD) is available throughout the curve. Figure 44-3A, \( K \) Value for Crest Vertical Curve (Stopping Sight Distance – Passenger Car), provides the \( K \) value for the design speed where \( S < L \). The following discusses the application of the \( K \) value.

1. **Passenger Car.** The \( K \) value is calculated by assuming \( h_1 = 3.5 \text{ ft}, \ h_2 = 2 \text{ ft}, \) and \( S \) = SSD in the basic equation for a crest vertical curve (Equation 44-3.1). The value represents the lowest acceptable sight distance on a facility. However, every reasonable effort should be made to provide a design in which the \( K \) value is greater than the value shown, where practical.

Where the stopping sight distance is greater than or equal to the vertical curve length, any of the following methods may be used to check the stopping sight distance.

a. **Using \( K \) Value.** The \( K \) value provided is greater than or equal to the \( K \) value required and there are no changes to \( G_1 \) or \( G_2 \) in Figure 44-3A(1), Crest Vertical Curve Stopping Sight Distance Using \( K \) Value.

b. **Using Equation.** Equation 44-3.3 shown above is only valid if there are no other vertical curves or angular breaks in the area shown in Figure 44-3A(1).

c. **Using the AASHTO Policy on Geometric Design of Highways and Streets.**

d. **Checking Graphically.** The eye should be placed at 3.5 ft above the pavement and the height of the object at 2 ft. The distance between the eye and the object that is unobstructed (by the road, backslope of a cut section, guardrail, etc.) is the stopping sight distance provided. It is necessary to check it in both directions for a 2-lane highway.
If the stopping sight distance provided exceeds that required (even though the $K$ value provided is less than the $K$ value required), the $K$ value will be treated as a Level Three design exception item instead of Level One.

If the $K$ value provided exceeds the $K$ value required, it is not necessary to perform either the equation check or the graphical check even though $S \geq L$.

2. **Truck.** The higher eye height for a truck, 7.6 ft, offsets the longer stopping distance required on a vertical curve. Therefore, the $K$ value for truck stopping sight distance need not be checked.

3. **Minimum Length.** The minimum length of a crest vertical curve in feet should be $3V$, where $V$ is the design speed in mph, unless existing conditions make it impractical to use the minimum-length criteria.

### 44-3.01(02) Decision Sight Distance

It may sometimes be warranted to provide decision sight distance in the design of a crest vertical curve. Section 42-2.0 discusses candidate sites and provides design values for decision sight distance. These $S$ values should be used in the basic equation for a crest vertical curve (Equation 44-3.1). In addition, the following will apply.

1. **Height of Eye ($h_1$).** For a passenger car, $h_1$ is 3.5 ft

2. **Height of Object ($h_2$).** Decision sight distance, is often predicated upon the same principles as stopping sight distance; i.e., the driver needs sufficient distance to see a 2-ft-height object.

3. **Passenger Car.** Figure 44-3B, $K$ Value for Crest Vertical Curve (Decision Sight Distance – Passenger Car), provides the $K$ value using the decision sight distance shown in Section 42-2.0.

### 44-3.01(03) Drainage

Drainage should be considered in the design of a crest vertical curve where a curbed section or concrete barrier is used. Drainage problems are minimized if the crest vertical curve is sharp enough so that a minimum longitudinal grade of at least 0.3% is reached at a point about 50 ft from either side of the apex. To ensure that this objective is achieved, the length of the vertical curve
should be based upon a $K$ value of 167 or less. For a crest vertical curve in a curbed section where this $K$ value is exceeded, the drainage design should be evaluated near the apex.

For an uncurved roadway section, drainage should not be a problem at a crest vertical curve. However, it is desirable to provide a longitudinal gradient of at least 0.15% at points about 50 ft on either side of the high point. To achieve this, $K$ must equal 300 or less.

See Part IV for more information on drainage.

### 44-3.02 Sag Vertical Curve

A sag vertical curve is in the shape of a parabola. It is designed to allow the vehicular headlights to illuminate the roadway surface (i.e., height of object = 0 ft) for a given distance $S$. A headlight height, $h_3$, of 2 ft, and a 1-deg upward divergence of the light beam from the longitudinal axis of the vehicle are assumed.

#### 44-3.02(01) Stopping Sight Distance

These assumptions yield the following equations for determining the minimum length of a sag vertical curve. If the stopping sight distance, $S$, is less than the vertical curve length, $L$,

$$L = \frac{AS^2}{400 + 3.5S}$$

(Equation 44-3.4)

If the stopping sight distance, $S$, is greater than or equal to the vertical curve length, $L$,

$$L = 2S - \frac{400 + 3.5S}{A}$$

(Equation 44-3.5)

where:

$L$ = length of vertical curve, ft  
$A$ = algebraic difference between the two tangent grades, %  
$S$ = sight distance, ft  
$K$ = horizontal distance needed to produce a 1% change in gradient

The length of the sag vertical curve will depend upon $A$ for the specific curve and upon the selected sight distance and headlight height. The following discusses the selection of these values.
The principal control in the design of a sag vertical curve is to ensure that, at a minimum, stopping sight distance (SSD) is available for headlight illumination throughout the curve. Figure 44-3C, $K$ Value for Sag Vertical Curve (Stopping Sight Distance – Passenger Car), provides the $K$ value for the design speed where $S < L$. The following discusses the application of the $K$ value.

1. **Passenger Car.** The $K$ value is calculated by assuming $h_3 = 2$ ft and $S = SSD$ in the equation for a sag vertical curve (Equation 44-3.4). The value represents the lowest acceptable sight distance on a facility. However, every reasonable effort should be made to provide a design in which the $K$ value is greater than the value shown, where practical.

Where the stopping sight distance is greater than or equal to the vertical curve length, any of the following methods may be used to check the stopping sight distance.

a. **Using $K$ Value.** The $K$ value provided is greater than or equal to the $K$ value required, and there are no changes to $G_1$ or $G_2$ as shown in Figure 44-3C(1), Sag Vertical Curve Stopping Sight Distance Using $K$ Value.

b. **Using Equation.** Equation 44-3.5 shown above is only valid if there are no other vertical curves or angular breaks in the area shown in Figure 44-3C(1).

c. **Using the AASHTO Policy on Geometric Design of Highways and Streets.**

d. **Checking Graphically.** The headlight should be placed at 2 ft above the pavement and the height of the object at 0 ft. The light beam is assumed at a 1-deg upward divergence from the longitudinal axis of the vehicle. The distance between the headlight and the object that is unobstructed (by the road, backslope of a cut section, guardrail, etc.) is the stopping sight distance provided. It is necessary to check it in both directions for a 2-lane highway.

If the stopping sight distance provided exceeds that required (even though the $K$ value provided is less than the $K$ value required), the $K$ value will be treated as a Level Three design exception item instead of Level One.

2. **Truck.** The higher headlight height for a truck, 4 ft, offsets the longer stopping distance required on a vertical curve. Therefore, the $K$ value for truck stopping sight distance need not be checked.

3. **Minimum Length.** The minimum length of a sag vertical curve in feet should be $3.2V$, where $V$ is the design speed in mph, unless existing conditions make it impractical to use the minimum length criteria.

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One exception to this minimum length may apply in a curbed section. If the sag is in a sump, the use of the minimum-length criteria may produce longitudinal slopes too flat to drain the stormwater without exceeding the criteria for the limits of ponding on the travel lane.

**44-3.02(02) Decision Sight Distance**

It may sometimes be warranted to provide decision sight distance in the design of a sag vertical curve. Section 42-2.0 discusses candidate sites and provides design values for decision sight distance. These $S$ values should be used in the equation for a sag vertical curve (Equation 44-3.5). The height of headlights, $h_3$, is 2 ft. Figure 44-3D, $K$ Value for Sag Vertical Curve (Decision Sight Distance – Passenger Car), provides the $K$ value using decision sight distance.

**44-3.02(03) Drainage**

Drainage should be considered in the design of a sag vertical curve where a curbed section or concrete barriers are used. Drainage problems are minimized if the sag vertical curve is sharp enough so that both of the following criteria are met.

1. A minimum longitudinal grade of at least 0.3% is reached at a point about 50 ft from either side of the low point.

2. There is at least a 0.25-ft elevation differential between the low point in the sag and the two points 50 ft to either side of the low point.

To ensure that the first objective is achieved, the length of the vertical curve should be based upon a $K$ value of 167 or less. For a sag vertical curve in a curbed section where this $K$ value is exceeded, the drainage design should be more carefully evaluated near the low point. For example, it may be necessary to install flanking inlets on either side of the low point.

For an uncurbed roadway section, drainage should not be a problem at a sag vertical curve. However, it is desirable to provide a longitudinal gradient of at least 0.15% at points about 50 ft on either side of the low point. To achieve this, $K$ must equal 300 or less.

See Part IV for more information on drainage.
44-3.02(04) Sight Distance at Undercrossing

Sight distance on a highway through a grade separation should be at least as long as the minimum stopping sight distance and preferably longer. Design of the vertical alignment is the same as at any other point on the highway except where a sag vertical curve underpasses a structure, as shown in Figure 44-3D, K Value for Sag Vertical Curve (Decision Sight Distance – Passenger Car). While not a frequent problem, the structure fascia may cut the line of sight and limit the sight distance to less than that otherwise attainable. It is practical to provide the minimum length of sag vertical curve at a grade separation structure. Where the recommended grades are exceeded, the sight distance should not be reduced below the minimum value for stopping sight distance.

The available sight distance should sometimes be checked at an undercrossing, such as at a two-lane undercrossing without ramps, where it would be desirable to provide passing sight distance. Such a check is best made graphically on the profile, but may be performed through computations.

The equations for sag vertical curve length at an undercrossing are as follows.

1. Sight distance, \( S \), greater than vertical curve length, \( L \),

\[
L = 2S - \left\{ \frac{800[C - 0.5(h_1 + h_2)]}{A} \right\} 
\]  
(Equation 44-3.6)

2. Sight distance, \( S \), less than or equal to vertical curve length, \( L \),

\[
L = \frac{AS^2}{800[C - 0.5(h_1 + h_2)]} 
\]  
(Equation 44-3.7)

For both equations, where:

\( L \) = length of vertical curve, ft  
\( S \) = sight distance, ft  
\( A \) = algebraic difference in grades, %  
\( C \) = vertical clearance, ft  
\( h_1 \) = height of eye, ft  
\( h_2 \) = height of object, ft

Using an eye height of 7.6 ft for a truck driver and an object height of 2 ft for the taillights of a vehicle, the following equation can be derived.
3. Sight distance, \( S \), greater than vertical curve length, \( L \),

\[
L = 2S - \frac{800(C - 5)}{A}
\]  
(Equation 44-3.8)

4. Sight distance, \( S \), less than or equal to vertical curve length, \( L \),

\[
L = \frac{AS^2}{800(C - 5)}
\]  
(Equation 44-3.9)

**44-3.03 Vertical-Curve Computations**

The following will apply to the mathematical design of a vertical curve.

1. **Definitions.** Figure **44-3E**, Vertical-Curve Definitions, provides the common terms and definitions used in vertical-curve computations.

2. **Measurements.** All measurements for a vertical curve are made on the horizontal or vertical plane, not along the profile grade. With the simple parabolic curve, the vertical offsets from the tangent vary as the square of the horizontal distance from the PVC or PVT. Elevations along the curve are calculated as proportions of the vertical offset at the point of vertical intersection (PVI). The necessary formulas for computing the vertical curve are shown in Figure **44-3F**, Symmetrical Vertical-Curve Equations. Figure **44-3G**, Vertical-Curve Computations (Example 44-3.1), provides an example of how to use these formulas.

3. **Unsymmetrical Vertical Curve.** It may be necessary to use an unsymmetrical vertical curve to obtain clearance on a structure or to satisfy some other design feature. This curve is similar to the parabolic vertical curve, except the curve does not vary symmetrically about the PVI. The necessary formulas for computing the unsymmetrical vertical curve are shown in Figure **44-3H**, Unsymmetrical Vertical-Curve Equations.

4. **Vertical Curve Through Fixed Point.** A vertical curve often must be designed to pass through an established point. For example, it may be necessary to tie into an existing transverse road or to clear an existing structure. See Figure **44-3 I**, Vertical-Curve Computations. Figure **44-3J**, Vertical-Curve Computations (Example 44-3.2), illustrates an example of how to use these formulas.
** PRACTICE POINTERS **

The profile grade should not be set too low. Field complaints about the profile grade having been set too low are much more common than complaints about it having been set too high.

The $K$ values for vertical curves should not be shown on the plans.

### 44-4.0 VERTICAL CLEARANCE

See Figure 44-4A, Minimum Vertical Clearance (New Construction or Reconstruction). Chapter 53 provides additional information. Chapters 54 through 56 provide vertical-clearance information for an existing highway.

### 44-5.0 DESIGN PRINCIPLES AND PROCEDURE

#### 44-5.01 General Controls for Vertical Alignment

As discussed elsewhere in this Chapter, the design of vertical alignment involves, to a large extent, complying with specific limiting criteria. These include maximum and minimum grades, sight distance at a vertical curve, and vertical clearance. The following design principles and controls should be considered which will determine the overall safety of the facility and will enhance the aesthetic appearance of the highway. These design principles for vertical alignment include the following.

1. **Consistency.** Use a smooth grade line with gradual changes, consistent with the type of highway and character of terrain, rather than a line with numerous breaks and short lengths of tangent grades.

2. **Environmental Impact.** Vertical alignment should be properly coordinated with environmental impact (e.g., encroachment onto wetlands). The Office of Environmental Services is responsible for evaluating environmental impacts.

3. **Long Grade.** On a long ascending grade, it is preferable to place the steepest grade at the bottom and flatten the grade near the top.
4. **Intersection.** Maintain moderate grades through an intersection to facilitate turning movements. See Chapter 46 for specific information on vertical alignment through an intersection.

5. **Roller Coaster.** The roller-coaster type of profile should be avoided. It may be proposed in the interest of economy, but it is aesthetically undesirable and may be hazardous.

6. **Broken-Back Curvature.** Avoid a broken-back grade line of two crest or sag vertical curves separated by a short tangent. One long vertical curve is more desirable.

7. **Coordination with Natural or Man-Made Feature.** The vertical alignment should be properly coordinated with the natural topography, available right of way, utilities, roadside development, or natural or man-made drainage patterns.

8. **Cut Section.** A sag vertical curve should be avoided in a cut section unless adequate drainage can be provided.

---

**44-5.02 Coordination of Horizontal and Vertical Alignment**

Horizontal and vertical alignment should not be designed separately, especially for a project on new alignment. Their importance demands that the interdependence of the two highway design features be carefully evaluated. This will enhance highway safety and improve the facility’s operation. The following should be considered in the coordination of horizontal and vertical alignment.

1. **Balance.** Curvature and grades should be in proper balance. Maximum curvature with flat grades or flat curvature with maximum grades does not achieve this desired balance. A compromise between the two extremes produces the best design relative to safety, capacity, ease, and uniformity of operations and a pleasing appearance.

2. **Coordination.** Vertical curvature superimposed upon horizontal curvature (i.e., vertical and horizontal PIs at approximately the same station) results in a more pleasing appearance and reduces the number of sight-distance restrictions. Successive changes in profile not in combination with the horizontal curvature may result in a series of humps visible to the driver for some distance, which may produce an unattractive design. However, sometimes superimposing the horizontal and vertical alignment must be tempered somewhat by Items 3 and 4 as follows.
3. **Crest Vertical Curve.** Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This is undesirable because the driver cannot perceive the horizontal change in alignment, especially at night when headlight beams project straight ahead into space. This problem can be avoided if the horizontal curvature leads the vertical curvature or by using design values which well exceed the minimums.

4. **Sag Vertical Curve.** A sharp horizontal curve should not be introduced at or near the low point of a pronounced sag vertical curve or at the bottom of a steep vertical grade. Because visibility to the road ahead is foreshortened, only flat horizontal curvature will avoid an undesirable, distorted appearance. At the bottom of a long grade, vehicular speeds often are higher, particularly for trucks, and erratic operations may occur, especially at night.

5. **Passing Sight Distance.** The need for frequent passing opportunities and a higher percentage of passing sight distance may sometimes supersede the desirability of combining horizontal and vertical alignment. It may be necessary to provide a long tangent section to secure sufficient passing sight distance.

6. **Intersection.** At an intersection, horizontal and vertical alignment should be as flat as practical to provide a design which produces sufficient sight distance and gradients for vehicles to slow or stop. See Chapter 46.

7. **Divided Highway.** On a divided facility with a wide median, it is frequently advantageous to provide independent alignments for the two one-way roadways. Where traffic justifies a divided facility, a superior design with minimal additional cost can result from the use of independent alignments.

8. **Residential Area.** The alignment should be designed to minimize nuisance factors to a neighborhood. A depressed facility makes the highway less visible and reduces the noise to adjacent residents. Minor adjustment to the horizontal alignment may increase the buffer zone between the highway and residential area.

9. **Aesthetics.** The alignment should be designed to enhance attractive scenic views of rivers, rock formations, parks, golf courses, etc. The highway should head into rather than away from those views that are considered to be aesthetically pleasing. The highway should fall towards those features of interest at a low elevation and rise toward those features which are best seen from below or in silhouette against the sky.
44-5.03 Profile-Grade Line

44-5.03(01) General

The profile-grade line is the roadway geometric characteristic which has the greatest impact on a facility’s costs, aesthetics, safety, and operation. The profile grade is a series of tangent lines connected by parabolic vertical curves. It is placed along the roadway centerline of an undivided facility or on the two pavement centerlines of a divided facility.

The designer must evaluate many factors in establishing the profile-grade line. These include the following:

1. maximum and minimum grades;
2. sight-distance criteria;
3. earthwork balance;
4. bridge or drainage structure;
5. high-water level;
6. drainage considerations;
7. water-table elevations;
8. highway intersection or interchange;
9. snow drifting;
10. railroad-highway crossing;
11. types of soil;
12. adjacent land use and values;
13. highway safety;
14. coordination with other geometric features (e.g., cross section);
15. topography or terrain;
16. truck performance;
17. right of way;
18. utilities;
19. urban or rural location;
20. aesthetics and landscaping;
21. construction costs;
22. environmental impacts;
23. driver expectations;
24. airport flight paths (e.g., grades and lighting); and
25. pedestrian and handicapped accessibility.

The following discusses the establishment of the profile-grade line in more detail.
44-5.03(02) Earthwork Balance

Where practical and where consistent with other project objectives, the profile-grade line should be designed to provide a balance of earthwork. This should not be achieved, however, at the expense of smooth grade lines and sight-distance requirements at a vertical curve. Ultimately, a project-by-project assessment will determine whether a project will be borrow, waste, or balanced.

The following should be considered in earthwork balance.

1. **Basic Approach.** The best approach to laying grade and balancing earthwork is to provide a significant length of roadway in embankment, to limit the number and amount of excavation areas. Long lengths of roadway in excavation with several short balance distances should be avoided.

2. **Urban or Rural.** Earthwork balance is a practical objective only in a rural area. In an urban area, other project objectives (e.g., limiting right-of-way impacts) have a higher priority than balancing earthwork. Excavated materials from an urban project are often unsuitable for embankments.

3. **Borrow Sites.** The availability and quality of borrow sites in the project vicinity will impact the desirability of balancing the earthwork.

4. **Mass Diagram.** A mass diagram illustrates the accumulated algebraic sum of material within the project limits. Such a diagram is useful in balancing earthwork and calculating haul distances and quantities. The mass diagram may indicate the following:
   a. the most economical procedure for disposing of excavated material,
   b. whether material should be moved backward or forward, or
   c. whether borrowing or wasting is more economical than achieving earthwork balance.

   A mass diagram is not prepared by the designer. It may be prepared and used by the contractor for construction operation.

5. **Balance Length.** A balance length is 2000 ft or longer. For an interchange, the balance points should be selected to incorporate the entire interchange.
6. Earthwork Computations. Chapter 17 discusses the proper methods to compute and record the project earthwork quantities.

44-5.03(03) Soils

The type of earth material encountered often influences the grade line at a certain location. If rock is encountered, for example, it may be more economical to raise the grade and reduce the rock excavation. Soils which are unsatisfactory for embankment or cause a stability problem in a cut area may also be determining factors in establishing a grade line. The development of the profile grade should be coordinated with the Office of Materials Management, which will conduct a soils survey.

44-5.03(04) Drainage and Snow Drifting

The profile-grade line should be compatible with the roadway drainage design and should minimize snow drifting problems. The following will apply.

1. Culvert. The roadway elevation should satisfy the Department criteria for minimum cover at a culvert and minimum freeboard above the head water level at a culvert. See Part IV for more information on culvert design.

2. Coordination with Geometrics. The profile-grade line must reflect compatibility between drainage design and roadway geometrics. These include the design of sag and crest vertical curves, spacing of inlets on a curbed facility, impacts on adjacent properties, superelevated curves, intersection design elements, and interchange design elements. For example, a sag vertical curve should be avoided in a cut section, and a long crest vertical curve should be avoided on a curbed pavement.

3. Snow Drifting. Where practical, the profile-grade line should be at least 3 ft above the natural ground level to prevent snow from drifting onto the roadway and to promote snow blowing off the roadway.

4. Water Table. The profile-grade line should be established such that the top of the subgrade elevation should be not less than 2 ft above the water table at all points along the cross section within the paved roadway surface. The elevation of the water table can be found in the Geotechnical Report. If it is not practical to provide the 2-ft clearance, the designer should meet with the Pavement Engineering Office manager and geotechnical engineer to develop an alternative solution.
44-5.03(05) Erosion Control

To minimize erosion, the following should be considered relative to the grade line.

1. Minimize the number of deep cuts and high fill sections.
2. Conform to the contour and drainage patterns of the area.
3. Make use of natural land barriers and contours to divert runoff and confine erosion and sedimentation.
4. Minimize the amount of disturbance.
5. Make use of existing vegetation.
6. Reduce slope length and steepness and ensure that erosion is confined to the right of way and does not deposit sediment on or erode away adjacent land.
7. Avoid locations having high base erosion potential.
8. Avoid cut or fill sections in a seepage area.

44-5.03(06) Bridge

The design of the profile-grade line must be coordinated with each bridge within the project limits. The following will apply:

1. **Vertical Clearance.** The criteria in Chapters 53 and 56 and Section 44-3.0 must be satisfied. In laying the preliminary grade line, an element in determining available vertical clearance is the assumed structure depth. This will be based on the structure type, span lengths, and depth/span ratio. For preliminary design, a 20-ft to 21-ft distance should be assumed between the finished grade of the roadway and the finished grade of the bridge deck. For final design, the designer must coordinate with the bridge designer to determine the roadway- and bridge-grade lines.

2. **Bridge Over Water.** Where the proposed facility will cross a body of water, the bridge elevation must be consistent with the necessary waterway opening to satisfy the Department’s hydraulic requirements. The designer must coordinate with the Production
Management Division’s Hydraulics Team and the bridge designer to determine the approach-roadway elevation to complement the necessary bridge elevation.

3. **Railroad Bridge.** A proposed facility over a railroad must satisfy the applicable criteria (e.g., vertical clearances, structure type, and depth). See Chapter 69 for more information.

4. **Highway Under Bridge.** Where practical, the low point of a roadway sag vertical curve should not be within the shadow of the bridge. This will help minimize ice accumulations, and it will reduce the ponding of water which may weaken the earth foundation beneath the bridge. To achieve these objectives, the low point of a roadway sag should be approximately 100 ft from the bridge.

5. **High Embankment.** The impacts of high embankment on a structure should be considered. This will increase the span length thus increasing structure costs.

6. **Low Point.** It is desirable to locate the low point of a sag vertical curve off the bridge deck.

**44-5.03(07) Distance Between Vertical Curves**

A desirable objective on a rural facility is to provide at least 1500 ft between two successive PVIs. This objective applies only to a project which has a considerable length where implementation is judged to be practical.

**44-5.03(08) Ties with Existing Highways**

A smooth transition is needed between the proposed profile grade line of the project and the existing grade line of an adjacent highway section. The existing grade line should be considered for a sufficient distance beyond the beginning or end of a project to ensure adequate sight distance. A connection should be made which is compatible with the design speed of the new project and which can be used if the adjoining road section is reconstructed.
Critical Lengths of Grade for Design, Assumed Typical Heavy Truck of 200 lb/hp, Entering Speed = 70 mph

Figure 44-1A
Critical Lengths of Grade Using an Approach Speed of 55 mph for Typical Recreational Vehicle

Figure 44-1B
Notes:

1. For vertical curves where the two tangent grades are in the same direction (both upgrades or both downgrades), 50% of the curve length will be part of the length of grade.

2. For vertical curves where the two tangent grades are in opposite directions (one grade up and one grade down), 25% of the curve length will be part of the length of the grade.
CRITICAL LENGTH OF GRADE CALCULATIONS
(Example 44-1.3)

Figure 44-1D
<table>
<thead>
<tr>
<th>DESIGN ELEMENT</th>
<th>DESIRABLE</th>
<th>MINIMUM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane Width</td>
<td>12 ft</td>
<td>Same as that required for through lane</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>Same as approach roadway</td>
<td>Freeway: Same as approach roadway (1) Non-Freeway: 4 ft paved</td>
</tr>
<tr>
<td>Cross Slope on Tangent</td>
<td>3%</td>
<td>2%</td>
</tr>
<tr>
<td>Beginning of Full-Width Lane</td>
<td>Near the PVC of the vertical curve preceding the grade.</td>
<td>At the PVT of the grade.</td>
</tr>
<tr>
<td>End of Full-Width Lane (2)</td>
<td>To where truck has reached highway design speed or 55 mph, whichever is lower.</td>
<td>To where truck has reached 10 mph below highway design speed or 45 mph, whichever is lower.</td>
</tr>
<tr>
<td>Entering Taper</td>
<td>100 ft</td>
<td>100 ft</td>
</tr>
<tr>
<td>Exiting Taper</td>
<td>50:1</td>
<td>500 ft</td>
</tr>
<tr>
<td>Minimum Full-Width Length</td>
<td>n/a</td>
<td>1000 ft</td>
</tr>
</tbody>
</table>

Notes:

(1) **On a reconstruction project, a 6-ft shoulder may be used.**

(2) **Use Figure 44-2B to determine truck deceleration rate. Use Figure 44-2C to determine truck acceleration rate. Also, see discussion in Section 44-2.03.**

**DESIGN CRITERIA FOR CLIMBING LANE**

**Figure 44-2A**
Speed-Distance Curves for a Typical Heavy Truck of 200 lb/hp for Deceleration on Upgrades

Figure 44-2B
Speed-Distance Curves for Acceleration of a Typical Heavy Truck of 200 lb/hp on Ugrades and Downgrades

Figure 44-2C
<table>
<thead>
<tr>
<th>DESIGN SPEED (mph)</th>
<th>ROUNDED SSD FOR DESIGN ¹ (ft)</th>
<th>CALCULATED K VALUE ²</th>
<th>K VALUE ROUNDED FOR DESIGN</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>115</td>
<td>80</td>
<td>6.1</td>
</tr>
<tr>
<td>20</td>
<td>155</td>
<td>115</td>
<td>11.1</td>
</tr>
<tr>
<td>25</td>
<td>200</td>
<td>155</td>
<td>18.5</td>
</tr>
<tr>
<td>30</td>
<td>250</td>
<td>200</td>
<td>29.0</td>
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<td>35</td>
<td>305</td>
<td>250</td>
<td>43.1</td>
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<td>60.1</td>
</tr>
<tr>
<td>45</td>
<td>425</td>
<td>360</td>
<td>83.7</td>
</tr>
<tr>
<td>50</td>
<td>495</td>
<td>425</td>
<td>113.5</td>
</tr>
<tr>
<td>55</td>
<td>570</td>
<td>495</td>
<td>150.6</td>
</tr>
<tr>
<td>60</td>
<td>645</td>
<td>570</td>
<td>192.8</td>
</tr>
<tr>
<td>65</td>
<td>730</td>
<td>645</td>
<td>246.9</td>
</tr>
<tr>
<td>70</td>
<td>820</td>
<td>730</td>
<td>312.6</td>
</tr>
</tbody>
</table>

Notes:

1. Stopping sight distance (SSD) is from Figure 42-1A.

2. The K value is calculated using the rounded value for design stopping sight distance, eye height of 3.5 ft, and object height of 2 ft.

3. If curbs are present, and K > 167, proper pavement drainage should be ensured near the high point of the curve.

**K VALUE FOR CREST VERTICAL CURVE**
(Stopping Sight Distance – Passenger Car)

Figure 44-3A
NO GRADE CHANGES *

* No other vertical curves or angular breaks within this zone.

STOPPING SIGHT DISTANCE CHECK USING K-VALUES,
CREST VERTICAL CURVE

Figure 44-3A(1)
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Avoidance Maneuver A (Stop on Rural Road)</th>
<th>Avoidance Maneuver B (Stop on Urban Road)</th>
<th>Avoidance Maneuver C (Speed/Path/Direction Change on Rural Road)</th>
<th>Avoidance Maneuver D (Speed/Path/Direction Change on Suburban Road)</th>
<th>Avoidance Maneuver E (Speed/Path/Direction Change on Urban Road)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DSD (ft)</td>
<td>$K$ Value</td>
<td>DSD (ft)</td>
<td>$K$ Value</td>
<td>DSD (ft)</td>
</tr>
<tr>
<td>20</td>
<td>90</td>
<td>11</td>
<td>270</td>
<td>54</td>
<td>300</td>
</tr>
<tr>
<td>25</td>
<td>110</td>
<td>15</td>
<td>335</td>
<td>72</td>
<td>375</td>
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<tr>
<td>30</td>
<td>220</td>
<td>23</td>
<td>490</td>
<td>112</td>
<td>450</td>
</tr>
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<td>35</td>
<td>590</td>
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<tr>
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<td>395</td>
<td>73</td>
<td>800</td>
<td>297</td>
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</tr>
<tr>
<td>50</td>
<td>465</td>
<td>100</td>
<td>910</td>
<td>384</td>
<td>750</td>
</tr>
<tr>
<td>55</td>
<td>535</td>
<td>133</td>
<td>1030</td>
<td>492</td>
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<td>60</td>
<td>610</td>
<td>173</td>
<td>1150</td>
<td>613</td>
<td>990</td>
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<tr>
<td>65</td>
<td>695</td>
<td>224</td>
<td>1275</td>
<td>754</td>
<td>1050</td>
</tr>
</tbody>
</table>

**Notes:**

1. See Section 42-2.0 for decision sight distances (DSD).
2. The $K$ value is calculated using the rounded value for design decision sight distance, eye height of 3.5 ft, and object height of 2 ft.

$$K = \frac{DSD^2}{2158}$$

3. If curbs are present and $K > 167$, proper pavement drainage should be ensured near the high point of the curve.

**$K$ Value for Crest Vertical Curve**

(Decision Sight Distance – Passenger Car)

*Figure 44-3B*
<table>
<thead>
<tr>
<th>DESIGN SPEED (mph)</th>
<th>ROUNDED SSD FOR DESIGN</th>
<th>CALCULATED K VALUE</th>
<th>K VALUE ROUNDED FOR DESIGN</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>115</td>
<td>16.5</td>
<td>17</td>
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<td>200</td>
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<td>37</td>
</tr>
<tr>
<td>35</td>
<td>250</td>
<td>49.0</td>
<td>49</td>
</tr>
<tr>
<td>40</td>
<td>305</td>
<td>63.4</td>
<td>64</td>
</tr>
<tr>
<td>45</td>
<td>360</td>
<td>78.1</td>
<td>79</td>
</tr>
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<td>50</td>
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<td>95.7</td>
<td>96</td>
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<td>157</td>
</tr>
<tr>
<td>70</td>
<td>730</td>
<td>180.3</td>
<td>181</td>
</tr>
</tbody>
</table>

Notes:

1. **Stopping sight distance (SSD) is from Figure 42-1A.**

2. **The K value is calculated using the rounded value for design stopping sight distance S and a headlight height of 2 ft.**

3. **If curbs are present and K > 167, proper drainage should be ensured near the low point of the curve.**

**K VALUE FOR SAG VERTICAL CURVE**  
(Stopping Sight Distance – Passenger Car)

Figure 44-3C
* No other vertical curves or angular breaks within this zone.

STOPPING SIGHT DISTANCE CHECK USING K-VALUES,
SAG VERTICAL CURVE

Figure 44-3C(1)
### Avoidance Maneuver A (Stop on Rural Road)
- Design Speed: 20 mph
- DSD (ft): 90
- K Value: 25
- DSD (ft): 270
- K Value: 68

### Avoidance Maneuver B (Stop on Urban Road)
- Design Speed: 25 mph
- DSD (ft): 110
- K Value: 39
- DSD (ft): 335
- K Value: 100

### Avoidance Maneuver C (Speed/Path/Direction Change on Rural Road)
- Design Speed: 30 mph
- DSD (ft): 220
- K Value: 55
- DSD (ft): 490
- K Value: 131

### Avoidance Maneuver D (Speed/Path/Direction Change on Suburban Road)
- Design Speed: 40 mph
- DSD (ft): 330
- K Value: 86
- DSD (ft): 690
- K Value: 188

### Avoidance Maneuver E (Speed/Path/Direction Change on Urban Road)
- Design Speed: 50 mph
- DSD (ft): 465
- K Value: 124
- DSD (ft): 910
- K Value: 251

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Avoidance Maneuver A (Stop on Rural Road)</th>
<th>Avoidance Maneuver B (Stop on Urban Road)</th>
<th>Avoidance Maneuver C (Speed/Path/Direction Change on Rural Road)</th>
<th>Avoidance Maneuver D (Speed/Path/Direction Change on Suburban Road)</th>
<th>Avoidance Maneuver E (Speed/Path/Direction Change on Urban Road)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DSD (ft)</td>
<td>K Value</td>
<td>DSD (ft)</td>
<td>K Value</td>
<td>DSD (ft)</td>
</tr>
<tr>
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<td>90</td>
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<td>270</td>
<td>68</td>
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<td>610</td>
<td>166</td>
<td>1150</td>
<td>320</td>
<td>990</td>
</tr>
<tr>
<td>65</td>
<td>695</td>
<td>190</td>
<td>1275</td>
<td>355</td>
<td>1050</td>
</tr>
</tbody>
</table>

**Notes:**

1. *The K value is calculated using the rounded value for design decision sight distance and headlight height of 2 ft.*
   
   \[ K = \frac{DSD}{120 + 3.5S} \]

2. *If curbs are present and K > 167, proper pavement drainage should be ensured near the low point of the curve.*

---

**K VALUE FOR SAG VERTICAL CURVE**

*(Decision Sight Distance – Passenger Car)*

Figure 44-3D
SIGHT DISTANCE AT UNDERCROSSING

Figure 44-3D(1)
<table>
<thead>
<tr>
<th>ELEMENT</th>
<th>ABBREVIATION</th>
<th>DEFINITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point of Vertical Curvature</td>
<td>PVC</td>
<td>The point at which a tangent grade ends and the vertical curve begins.</td>
</tr>
<tr>
<td>Point of Vertical Tangency</td>
<td>PVT</td>
<td>The point at which the vertical curve ends and the tangent grade begins.</td>
</tr>
<tr>
<td>Point of Vertical Intersection</td>
<td>PVI</td>
<td>The point where the extension of two tangent grades intersect.</td>
</tr>
<tr>
<td>Grade</td>
<td>$G_1, G_2$</td>
<td>The rate of slope between two adjacent PVIs expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in feet for each 100 ft of horizontal distance. An upgrade in the direction of stationing is identified as plus (+). A downgrade is identified as minus (-).</td>
</tr>
<tr>
<td>External Distance</td>
<td>$M$</td>
<td>The vertical distance (offset) between the PVI and the roadway surface along the vertical curve.</td>
</tr>
<tr>
<td>Algebraic Difference in Grade</td>
<td>$A$</td>
<td>The value is the deflection in percent between two tangent grades.</td>
</tr>
<tr>
<td>Length of Vertical Curve</td>
<td>$L$</td>
<td>The horizontal distance in feet from the PVC to the PVT.</td>
</tr>
</tbody>
</table>

**VERTICAL-CURVE DEFINITIONS**

Figure 44-3E
\( M = \text{Mid-ordinate, feet} \)
\( Z = \text{Any tangent offset, feet} \)
\( L = \text{Horizontal length of vertical curve, feet} \)
\( X = \text{Horizontal distance from PVC or PVT to any ordinate } Z, \text{ feet} \)
\( G_1 \text{ and } G_2 = \text{Rates of grade, expressed algebraically, percent} \)

All expressions are to be calculated algebraically.

\[
PVI \text{ Elev} = PVC \text{ Elev} + \frac{LG_1}{200}
\]

\[
PVT \text{ Elev} = PVC \text{ Elev} + \frac{L(G_1 + G_2)}{200}
\]

\[
M = \frac{L(G_2 - G_1)}{800}
\]

For offset \( Z \) at distance \( X \) from PVC or PVT:

\[
Z = M \left( \frac{2X}{L} \right)^2 \quad \text{or} \quad Z = \frac{X^2(G_2 - G_1)}{200L}
\]

For slope \( S \), in percent, of a line tangent to any point on the vertical curve at distance \( X \) measured from the PVC:

\[
S = G_1 - \frac{X(G_1 - G_2)}{L}
\]

Calculate location and elevation of the high or low point on the curve:

\[
X_T = \frac{LG_1}{G_1 - G_2}
\]

Where \( X_T \) equals the horizontal distance from the PVC to the high or low point on the curve, feet.

\[
Elev = PVC \text{ Elev} - \frac{L(G_1)^2}{200(G_2 - G_1)}
\]

**SYMMETRICAL VERTICAL-CURVE EQUATIONS**

*Figure 44-3F*
**Example 44-3.1**

Given:

\[ G_1 = -1.75\% \]
\[ G_2 = +2.25\% \]

Elev. of PVI = 577.50
Station of PVI = 13+80
\( L = 500 \text{ ft} \)

Problem: Compute the grade for each 50-ft increment. Compute the low point station and elevation.

Solution:

1. Draw a diagram of the vertical curve and determine the station of the beginning (PVC) and the end (PVT) of the curve.

   **Beginning Station (PVC)** = PVI Sta. – 0.5\( L \) = (13+80) – (2+50) = 11+30
   **End Station (PVT)** = PVI Sta. + 0.5\( L \) = (13+80) + (2+50) = 16+30

2. Solve the vertical curve equations:

   \[
   M = \frac{(G_2 - G_1) L}{800} = \frac{(2.25 - (-1.75)) 500}{800} = 2.50 \text{ ft}
   \]

   \[
   Z = M \left( \frac{X}{L/2} \right)^2 = \frac{4MX^2}{L^2} = \frac{(4)(2.5)X^2}{250,000} = \frac{X^2}{25000}
   \]

3. Set up a table to show the vertical curve elevations at the 50-ft increments:

<table>
<thead>
<tr>
<th>Station</th>
<th>Inf.</th>
<th>Tangent Elevation</th>
<th>X</th>
<th>( X^2 )</th>
<th>Z</th>
<th>Grade Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>11+30 PVC</td>
<td>581.875</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>581.875</td>
<td></td>
</tr>
<tr>
<td>11+80</td>
<td>581.000</td>
<td>50</td>
<td>2500</td>
<td>0.100</td>
<td>581.100</td>
<td></td>
</tr>
<tr>
<td>12+30</td>
<td>580.125</td>
<td>100</td>
<td>10000</td>
<td>0.400</td>
<td>580.525</td>
<td></td>
</tr>
<tr>
<td>12+80</td>
<td>579.250</td>
<td>150</td>
<td>22500</td>
<td>1.125</td>
<td>580.375</td>
<td></td>
</tr>
<tr>
<td>13+30</td>
<td>578.385</td>
<td>200</td>
<td>40000</td>
<td>2.000</td>
<td>580.375</td>
<td></td>
</tr>
<tr>
<td>13+80 PVI</td>
<td>577.500</td>
<td>250</td>
<td>62500</td>
<td>3.125</td>
<td>580.625</td>
<td></td>
</tr>
<tr>
<td>14+30</td>
<td>578.675</td>
<td>200</td>
<td>40000</td>
<td>2.000</td>
<td>580.675</td>
<td></td>
</tr>
<tr>
<td>14+80</td>
<td>579.750</td>
<td>150</td>
<td>22500</td>
<td>1.125</td>
<td>580.875</td>
<td></td>
</tr>
<tr>
<td>15+30</td>
<td>580.875</td>
<td>100</td>
<td>10000</td>
<td>0.400</td>
<td>581.275</td>
<td></td>
</tr>
<tr>
<td>15+80</td>
<td>582.000</td>
<td>50</td>
<td>2500</td>
<td>0.100</td>
<td>582.700</td>
<td></td>
</tr>
<tr>
<td>16+30 PVT</td>
<td>583.125</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>583.125</td>
<td></td>
</tr>
</tbody>
</table>

4. Determine low point on curve:
therefore, the station at low point is

\[(11+30.00) + (2+18.75) = (13+48.75)\]

and the elevation of low point on curve is

\[\text{Elev. PVC} - \frac{L(G_1)^2}{(G_2 - G_1)200} = 581.875 - \frac{500(-1.75)^2}{[2.25 - (-1.75)]200} = 581.875 - 1.545 = 580.33\]

**VERTICAL-CURVE COMPUTATIONS**

(Example 44-3.1)

**Figure 44-3G**
\( P \) = Theoretical Point at PVI
\( M \) = Offset from the PVI to the curve, feet
\( Z \) = Any tangent offset, feet
\( L \) = Horizontal length of vertical curve, feet
\( L_1 \) = Horizontal distance from PVC to PVI, feet
\( L_2 \) = Horizontal distance from PVT to PVI, feet
\( X \) = Horizontal distance from PVC or PVT to any ordinate Z, feet
\( G_1 \) and \( G_2 \) = Rates of grade, expressed algebraically, percent

All expressions to be calculated algebraically, as follows:

\[
PVI \text{ ELEV} = PVC \text{ ELEV} + \frac{G_1 L_1}{100}
\]

\[
PVT \text{ ELEV} = PVC \text{ ELEV} + \frac{G_1 L_1}{100} + \frac{G_2 L_2}{100}
\]

\[
P \text{ ELEV} = PVC \text{ ELEV} + \frac{L_1}{L} \left( \frac{G_1 L_1 + G_2 L_2}{100} \right)
\]

\[
M = \left( \frac{L_1 L_2}{200 L} \right) (G_2 - G_1) = \frac{P \text{ ELEV} - PVI \text{ ELEV}}{2}
\]

For offset \( Z \) at distance \( X \) from PVC:

\[
Z = M \left( \frac{X}{L_1} \right)^2
\]

For offset \( Z \) at a distance \( X \) from PVT:

\[
Z = M \left( \frac{X}{L_2} \right)^2
\]

The high or low point on curve is calculated as follows:

If the high or low point occurs on the left portion of the curve:

\[
X_T = \frac{L_1}{L_2} \left( \frac{G_1 L}{G_2 - G_1} \right)
\]

Where \( X_T \) equals the horizontal distance from the PVC to the high or low point on the curve, feet.
If the high or low point occurs on the right portion of curve:

$$Elev \text{ of this Pt.} = PVC \text{ ELEV} - \frac{L_1}{L_2} \left[ \frac{L(G_1)^2}{(G_2 - G_1)200} \right]$$

If the high or low point occurs on the right portion of curve:

$$X_T = \frac{L_2}{L_1} \left( \frac{G_2 L}{G_2 - G_1} \right)$$

Where $X_T$ equals the horizontal distance from the PVC to the high or low point on the curve, feet.

$$Elev \text{ of this Pt.} = PV T \text{ ELEV} - \frac{L_2}{L_1} \left[ \frac{L(G_2)^2}{(G_1 - G_2)200} \right]$$

**UNSYMMETRICAL VERTICAL-CURVE EQUATIONS**

*Figure 44-3H*
TO PASS A VERTICAL CURVE THROUGH A GIVEN POINT P

\[ G_1 = \text{Grade In, \%} \]
\[ G_2 = \text{Grade Out, \%} \]
\[ A = \text{Algebraic difference in grades, \%} \]
\[ Z = \text{Vertical curve correction at point P, feet} \]
\[ X = \text{Distance from point P to PVC, feet} \]
\[ D = \text{Distance from point P to PVI, feet} \]
\[ L = \text{Length of vertical curve, feet} \]

**Given:** \( G_1, G_2, D \)

**Find:** Length of vertical curve

**Solution:**

1. Find algebraic difference in grades:
   \[ A = G_2 - G_1 \]

2. Find vertical curve correction at point P at distance \( x \) measured from PVC:
   \[ Z = X^2 \left( \frac{G_2 - G_1}{200L} \right) \]

3. From inspection of the above diagram:
   \[ \frac{L}{2} = X + D \text{, or } L = 2(X + D) \]

Substituting \( 2(X+D) \) for \( L \) and \( A \) for \( (G_2-G_1) \) yields:

\[ AX^2 = (-400ZX) + (-400DZ) = 0 \]
4. Solve for $X$ given the quadratic equation as follows:

$$X = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} = \frac{400Z \pm \sqrt{160,000Z^2 + 1600ADZ}}{2A}$$

Solving for $X$ will result in two answers. If both answers are positive, there are two solutions. If one answer is negative, it can be eliminated and only one solution exists.

5. Substitute $X$ and $D$ into the equation shown in Step 3 and solve for $L$.

Note: Two positive $X$ values will result in two solutions for $L$. Desirably, the solution that results in a longer $L$ should be used provided that it satisfies the stopping sight distance criteria based on the selected design speed and algebraic difference in grades. See Figures 44-3A and 44-3C).

**VERTICAL-CURVE COMPUTATIONS**

*Figure 44-3I*
Example 44-3.2

Given: Design Speed = 55 mph
\[ G_1 = -1.5\% \]
\[ G_2 = +2.0\% \]
\[ A = 3.5\% \]
PVI Station = 49+10
PVI elevation = 642.10

Problem: At Station 47+46, the new highway must pass under the center of an existing railroad which is at elevation 669.00 at the highway centerline. The railroad bridge that will be constructed over the highway will be 4 ft in depth, 20 ft in width and at right angles to the highway. What would be the length of the vertical curve that would provide a 16.5-ft clearance under the railroad bridge?

Solution:

1. Sketch the problem with known information.
Example 44-3.2 (continued)

2. Determine the station where the minimum 16.5-ft vertical clearance will occur (Point P):

From inspection of the sketch, the critical location is on the left side of the railroad bridge. The critical station is as follows:

\[
\text{Sta. P} = \text{Bridge Centerline Sta.} - \frac{1}{2} (\text{Bridge Width})
\]
\[
\text{Sta. P} = \text{Sta. (47+46)} - \frac{1}{2} (0+20)
\]
\[
\text{Sta. P} = \text{Sta. 47+36}
\]

3. Determine the elevation of Point P:

\[
\text{Elev. P} = \text{Elev. Top of Bridge} - \text{Bridge Depth} - \text{Clearance}
\]
\[
\text{Elev. P} = 669.00 - 4.00 - 16.5
\]
\[
\text{Elev. P} = 648.50
\]

4. Determine distance \(D\) from Point P to PVI:

\[
D = \text{STA. PVI} - \text{STA. P}
\]
\[
= (49+10) - (47+36) = 174 \text{ ft}
\]

5. Determine the tangent elevation at Point P:

\[
\begin{align*}
\text{Elev} &= \text{PVI Elev} - G_i \left( \frac{D}{100} \right) \\
\text{Elev} &= 642.10 - (-1.5) \left( \frac{174}{100} \right)
\end{align*}
\]

Elev. is 644.71

6. Determine the vertical curve correction \(Z\) at Point P:

\[
Z = \text{Elev. on Curve} - \text{Elev. on Tangent}
\]
\[
= 648.50 - 644.71 = 3.79 \text{ ft}
\]

7. Solve for \(X\) using equation from Figure 44-3 I, Step 4:

\[
X = \frac{400 Z \pm \sqrt{160,000 Z^2 + 1600 ADZ}}{2A}
\]
\[
X = \frac{400 (3.79) \pm \sqrt{(160,000)(3.79)^2 + 1600(3.5)(174)(3.79)}}{2(3.5)}
\]

8. Using Figure 44-3 I, Step 3, solve for \(L\):
\( X = 566.24 \text{ ft or } -133.10 \text{ ft} \) [Disregard negative value]

\[
L = 2(X + D) \\
L = 2(566.24 + 174) \\
L = 1480.48 \text{ ft}
\]

9. Determine if the solution meets the stopping sight distance for the 55-mph design speed.

From Figure 44-3C, the \( K \) value is 115.

The algebraic difference in grades is as follows:

\[
A = G_2 - G_1 = (+2.0) - (-1.5) = 3.5
\]

From Equation 44-3.2, the minimum length of vertical curve which meets the stopping sight distance is as follows:

\[
L = KA \\
= (115)(3.5) \\
= 402.50 \text{ ft}
\]

\( L \) of 1480.48 ft exceeds 402.50 ft., therefore the desirable stopping sight distance is satisfactory.

**VERTICAL-CURVE COMPUTATIONS**

*(Example 44.3-2)*

*Figure 44-3J*
<table>
<thead>
<tr>
<th>Type</th>
<th>Minimum Clearance (ft-in.)</th>
</tr>
</thead>
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<tr>
<td>Freeway Under Bridge</td>
<td>16’-6” (1.) (2.)</td>
</tr>
<tr>
<td>Arterial Under Bridge</td>
<td>16’-6” (1.) (3.)</td>
</tr>
<tr>
<td>Collector Under Bridge</td>
<td>14’-6” (1.)</td>
</tr>
<tr>
<td>Local Road Under Bridge</td>
<td>14’-6” (1.)</td>
</tr>
<tr>
<td>Roadway under Pedestrian Bridge</td>
<td>17’-6” (1.)</td>
</tr>
<tr>
<td>Roadway under Traffic Signal</td>
<td>17’-0” (1.) (4.)</td>
</tr>
<tr>
<td>Railroad under Roadway (Typical)</td>
<td>23’-0” (5.)</td>
</tr>
<tr>
<td>Roadway under Sign Truss</td>
<td>17’-6” (1.)</td>
</tr>
<tr>
<td>Non-Motorized-Vehicle-Use Facility under Bridge</td>
<td>10’-0” (6.)</td>
</tr>
</tbody>
</table>

Notes:

1. Value allows 6 in. for future resurfacing.
2. A 14’-6” clearance (including future resurfacing) may be used in an urban area where an alternative freeway facility with a 16’-0” clearance is available.
3. In a highly urbanized area, a minimum clearance of 14’-6” (including future resurfacing) may be provided if there is at least one route with a 16’-0” clearance.
4. Distance is measured from roadway surface to the bottom of signal at the bottom of the back plate or to the mast arm. See the INDOT Standard Drawings.
5. See Chapter Sixty-nine for additional information on a railroad under a roadway.
6. Value allows for clearance of a maintenance or emergency vehicle.

MINIMUM VERTICAL CLEARANCE,
NEW CONSTRUCTION OR RECONSTRUCTION

Figure 44-4A
CHAPTER 45

Cross-Section Elements

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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CHAPTER 45

CROSS-SECTION ELEMENTS

Chapter 53 provides numerical criteria for various cross-section elements for a new-construction or reconstruction project. Chapters 54 through 56 provide criteria for cross-section elements for an existing highway. This Chapter provides additional guidance which should be considered in the design of each cross-section element. The designer should also review the typical cross sections provided in Section 45.8.0.

45-1.0 ROADWAY SECTION

45-1.01 Travel Lane

45-1.01(01) Width

Travel-lane width can vary from 9 ft through 12 ft, depending upon the functional classification, traffic volume, design speed, rural or urban location, and project scope of work. The tables in Chapters 53 through 56 provide specific criteria for travel-lane width for these various conditions.

45-1.01(02) Cross Slope

Surface cross slopes are required for the proper drainage of through travel lanes on a tangent section. This reduces the hazard of wet pavement by quickly removing water from the surface, and reduces the likelihood of ponding. On a State highway, the following will apply for a tangent roadway section.

1. **2-Lane Highway.** The travelway lane pavement should be crowned at the centerline with a cross slope of 2% sloping away from the center.

2. **Divided Facility.** For two lanes in each direction, each roadway is crowned at the centerline with a cross slope of 2% sloping away from the center. For three or more lanes in each direction, the following will apply.
   a. **Three-Lane Section (New Construction or Reconstruction).** The pavement is crowned along the lane edge between the center lane and the lane adjacent to the
median, with the right two lanes sloping to the outside. The travel-lanes cross slopes should be 2%.

b. Three-Lane Section (Adding Lanes to Existing Facility). When adding new lanes either in the median or on the outside, the existing roadway crown is maintained. The added-travel-lane cross slope direction and rate will be the same as that of the adjacent travel lane. Where three lanes are sloped in the same direction, the third lane should be sloped at 3%.

c. Four-Lane Section. The travelway pavement should be crowned at the one-way-roadway centerline (i.e., two lanes on each side) with a cross slope of 2% sloping away from the center. Where three or more lanes are sloped in the same direction, the third and fourth lanes should be sloped at 3%.

d. Existing. For a roadway with 2-lanes sloped in one direction, increase the overlay depth by 3 in. on the inside edge to achieve a uniform 2% cross slope downward across both travelway lanes. For three lanes sloped in one direction, use a 3% cross slope for the outside lane. If the additional lane is to be added in the median, it should be sloped at 2% toward the median.

3. Bridge. For a new or reconstructed bridge, the cross slope will be 2% sloping away from the crown and will apply to the entire width from the crown to the face of the railing or curb. The crown across the bridge will be in the same location as the approaching roadway. An existing bridge to remain in place may retain an existing cross slope of 1.5%.

For a non-State highway, the travel-lanes cross slopes will vary depending upon the pavement surface and local practices. For a paved surface, the cross slope should be the same as for a State highway (2%). For a non-State facility with an aggregate surface, the cross slope should be 6%.

45-1.02 Shoulders or Curb Offsets

45-1.02(01) Definitions

The following definitions apply.

1. Shoulder. The portion of the roadway contiguous with the traveled way for accommodation of a stopped vehicle, for emergency use, or for lateral support of subbase, base, and surface courses.
2. **Usable-Shoulder Width.** The width of the shoulder that can be used by a driver for emergency parking or stopping. Figure 45-1A illustrates the definition of usable-shoulder width.

3. **Effective Usable-Shoulder Width.** This width is equal to the usable-shoulder width minus 1 ft. However, the effective usable-shoulder width cannot be less than the required paved-shoulder width.

4. **Curb Offset.** The term is used to define the distance between the edge of the travel lane and the face of curb.

**45-1.02(02) Functions**

A shoulder serves many functions. The wider the shoulder, the greater the benefits, including the following:

1. providing structural lateral support for the travelway;

2. increasing highway capacity;

3. encouraging uniform travel speed;

4. providing space for emergency or discretionary stops;

5. improving roadside safety by providing more recovery area for a run-off-the-road vehicle;

6. providing a sense of openness;

7. improving sight distance around a horizontal curve;

8. enhancing highway aesthetics;

9. facilitating maintenance operations (e.g., snow storage);

10. providing additional lateral clearance to a roadside appurtenance (e.g., guardrail, traffic signal);

11. facilitating pavement drainage;

12. providing space for pedestrian and bicycle use; and
13. providing space for a bus stop.

45-1.02(03) Width

Shoulder width will vary according to functional classification, traffic volume, urban or rural location, curbed or uncurbed facility, and project scope of work. The figures in Chapters 53 through 56 provide the paved- and usable-shoulder width criteria for these conditions. See Section 49-5.0 for shoulder width where guardrail is required.

45-1.02(04) Surface Type

For a new or reconstruction project on a State highway, the shoulder will be paved with asphalt or concrete. On a 3R or partial 3R project on a State highway, the shoulder should be paved. However, a sealed-aggregate shoulder may be appropriate. For a non-State highway, the shoulder should be paved. However, a sealed-aggregate or earth surface is acceptable.

45-1.02(05) Cross Slope

The cross slope of the shoulder varies according to the shoulder type and width. It should be the same across the full width of the usable shoulder. One exception is shown in Section 55-4.03(02) Item 4. The figures in Chapters 53 through 56 provide the cross slopes used for each classification. For a paved shoulder of 4 ft or narrower, the shoulder cross slope should be the same as that of the adjacent travel lane. See Figure 45-1A(1), Paved-Shoulder Cross Slope and Pavement Treatment, Tangent Section, with Underdrains; or Figure 45-1A(2), Paved-Shoulder Cross Slope and Pavement Treatment, Tangent Section, without Underdrains.

The following summarizes INDOT and local public agency practice:

1. Paved. The cross slope is 4%.
2. Curb Offset. The curb offset is paved and has the same cross slope as the adjacent travel lane, which is typically 2%.
3. Aggregate. The cross slope is 4 to 6%.
4. Earth. The cross slope is 6 to 8%.
45-1.02(06) Shoulder Corrugations [Rev. Feb. 2019]

Shoulder corrugations guidance has been moved to Section 502-2.0 Pavement Markings.

45-1.03 Auxiliary Lane

An auxiliary lane includes a left- or right-turn lane, acceleration or deceleration lane, or climbing lane. An auxiliary lane should be the same width as the adjacent travel lane, but not less than 1 ft narrower. The figures in Chapters 53 through 55 provide the specific width criteria for an auxiliary lane. The figures also provide the criteria for shoulder width adjacent to an auxiliary lane.

The cross slope for an auxiliary lane should be 1% greater than that of the adjacent through lane.

Chapter 46 provides additional information for a two-way left-turn lane.

45-1.04 Parking Lane (On-Street)

For an urban-area project, the designer must evaluate the demand for parking. Such parking needs will be accommodated by providing an off-street parking facility. Chapter 51 provides information on the design and layout of an off-street parking facility. If providing on-street parking along an urban street, the designer should evaluate the following.

1. **Warrants.** Adjacent land use may create the need to provide on-street parking along an urban street. A parking lane provides convenient access for a motorist to a business or residence. However, on-street parking reduces capacity, impedes traffic flow, may produce undesirable traffic operations, or may increase the accident potential. Therefore, a new parking lane should not be placed along a State highway. The designer should consider removing parking lanes on a State-highway reconstruction (4R) project, wherever practical. Removal of, or revising an existing on-street parking configuration will require concurrence from local officials and an official action by INDOT.

2. **Configuration.** The two types of on-street parking are parallel and angle parking. These are illustrated in Figure 45-1B. Parallel parking is the preferred arrangement where street space is limited and traffic capacity is a major factor. Angle parking provides more spaces per linear foot than parallel parking, but a greater street width is necessary for this design. The total entrance and exit time for parallel parking exceeds that required for angle parking. Parallel
parking also requires a vehicle to stop in the travel lane and await an opportunity to back into the parking space. However, the designer should also consider that angle parking requires the vehicle to back into the lane of travel where sight distance may be restricted by adjacent parked vehicles or where this maneuver may surprise an approaching motorist.

In selecting the parking configuration, the designer should evaluate the operational consequences of the selection. The designer should consider the backing maneuver required with angle parking. As indicated in Figure 45-1B, the parked car will require a certain distance $B$ to back out of its stall. Whether or not this is a reasonably safe maneuver will depend upon the number of lanes in each direction, lane width, operating speed, traffic volume during peak hours, parking demand, and turnover rate of parked vehicles.

On a new-construction project, only parallel parking should be provided. An existing facility with angle parking should be converted to parallel parking. Changes to existing on-street parking will require concurrence from local officials and an official action by INDOT.

3. **Stall Dimensions.** Figure 45-1B provides the width and length criteria for a parking stall for various configurations. The figure also indicates the number of stalls which can be provided for each parking configuration for a given curb length.

The figures in Chapters 53 and 55 provide parking-lane width for parallel parking. For angle parking, the parking lane width will be a combination of $A$ and $B$ as shown in Figure 45-1B, exclusive of the through travel lane. However, in a restricted area, a portion of the $B$ dimension may be required for the through travel lane, thereby reducing the actual parking-lane width. Figure 45-1C provides the recommended street width that should be considered with on-street parking.

Section 51-1.07 provides information on parking-stall dimensions for a accessible parking space.

4. **Cross Slope.** The cross slope of the parking lane will be 1% steeper than that of the adjacent travel lane, therefore 3%.

5. **Accessibility.** Section 51-1.07 provides the requirements for accessible on-street parking.

6. **Location.** In locating parking spaces, the designer should consider the following.

   a. Parking is prohibited within 20 ft of a crosswalk.

   b. Parking should be prohibited within 5 to 10 ft of the beginning of the curb radius at a
mid-block drive entrance.

c. Parking is prohibited within 50 ft of the nearest rail of a railroad-highway crossing.

d. Parking is prohibited within 15 ft of a fire hydrant.

e. Parking is prohibited within 30 ft on the approach leg to an intersection with a flashing beacon, stop sign, or traffic control signal. For a no-control or yield-controlled intersection, parking is not allowed within the intersection itself.

f. Parking is prohibited within 20 ft of the near side of a fire station drive entrance, and 75 ft from the entrance for the opposite side of the street.

g. Parking is prohibited on a bridge or within a highway tunnel.

h. Parking is prohibited along the same side or opposite a street excavation or obstruction if it would obstruct traffic.

i. Parking should be prohibited from areas designated by local traffic and enforcement regulations (e.g., near a school zone, loading zone, bus stop). See local ordinances for additional information on parking restrictions.

45-1.05 Curbs

Curbs are often used on an urban facility to retain the cut slope, control drainage, delineate the pavement edge, reduce right-of-way requirements, channelize vehicular movements, and improve aesthetics. In an urban area, curbs have a major benefit in containing the drainage within the pavement area and in channelizing traffic into and out of adjacent properties.

A curbed cross section is an appropriate design option in an outlying suburban or intermediate setting, or in an area undergoing or in imminent transition from rural-to-suburban land use, as well as in a low-speed or built-up urban setting. This clarification and latitude to expand opportunities for selection of a curbed cross-section is due in part to a desire by INDOT to plan each facility in context with existing and planned land-use characteristics.

45-1.05(01) Warrants For a Curbed Section
Selecting a curbed section or uncurbed section depends upon many variables, including vehicular speed, urban or rural location, drainage, and construction costs. The following discusses those factors which will determine whether or not a curbed section is warranted.

1. **Urban Location.** A curbed section is typically used in a Built-Up urban area due to restricted right of way, other constraints, and to better delineate travel lanes or parking lanes from pedestrian-use areas.

   A curbed section may be considered in a Suburban or Intermediate location for a design speed as high as 55 mph. The use of a curbed or uncurbed section will be made on a project-by-project basis, considering right-of-way constraints, drainage, pedestrian activity, channelization needs, drive access control, etc. This applies to new-construction, 4R, or 3R work in each functional classification other than freeway. The exceptions listed under Item 2 below for a rural location also apply to a high-speed Suburban facility.

2. **Rural Location.** The use of curbs is usually limited to conditions such as the following:
   a. where there is sufficient development along the highway and there is a need to channelize traffic into and out of properties;
   b. where it is absolutely necessary to control drainage;
   c. where restricted right-of-way provides insufficient space for roadside ditches;
   d. to lessen property impacts;
   e. to prevent soil erosion;
   f. the design speed is 55 mph or lower; or
   g. where otherwise deemed absolutely necessary.

Shoulders may be appropriate in a curbed cross section. However, it is acceptable practice not to provide a shoulder aside a curb for a design speed of 55 mph or lower. The appropriate figure in Chapter 53, 54, or 55 shows the shoulder width adjacent to a curb where a shoulder is used.
45-1.05(02) Types

There are two types of curbs, sloping and vertical. A sloping curb has a height of 4 in. or lower with a face batter no steeper than approximately one horizontal to one vertical. A vertical curb has a height of up to 6 in. with a face batter steeper than one horizontal to six vertical. The INDOT Standard Drawings illustrate the typical curb sections used by the Department, and provide details for these and other curb types.

1. **Sloping Curb.**
   a. Curb Height of 4 in. This curb height should be used where a curb is determined to be warranted and the design speed is 30 mph or higher. In a Suburban or Intermediate urban location, the curb should be located at the edge of the paved shoulder. The shoulder widths to be used in either of these locations are shown in Figures 53-6 through 53-9, and Figures 55-3E through 55-3H.

   b. Curb Height of 3½ in. This curb height should only be used by a local public agency in a residential area where curbs are determined to be warranted. It should not be used on an INDOT-maintained route. However, it may be used to reconstruct a local street disturbed by INDOT-facility construction.

2. **Vertical Curb.** A vertical curb is only used on a low-speed, urban Built-Up facility where the design speed is 25 mph or lower. A vertical curb may be used where the design speed reaches 45 mph, but only for drainage or curbed-section continuity.

Although a vertical curb may deflect a vehicle at a lower speed, it should not be used in lieu of guardrail as protection from an obstruction. Where vehicular encroachment is permissible, a sloping curb should be used.
45-1.05(03)  Curb Type Selection

1. **Materials.** Concrete curbs are used. However, for a project on an existing facility, asphalt curbing, not to exceed 4 in. in height, may be used under guardrail to control erosion. Asphalt curbing may also be used for a temporary island, temporary median within a construction zone, etc. Where snowplowing operations are conducted, asphalt curbing may be subject to severe damage or total removal. Therefore, it should not be used where damage from snowplows can be expected.

2. **Speed.** Vertical curbs are used only on a low-speed, urban facility where the design speed is 45 mph or lower. Preferably, curbs should not be used along a rural or high-speed urban highway with a design speed of 50 mph or higher. If curbs are deemed necessary, only sloping curbs located at the edges of the shoulders should be used on such a high-speed facility.

3. **Vehicular Encroachment.** Although at a lower speed a vertical curb may deflect a vehicle, it should not be used in lieu of guardrail as protection from a hazardous object. Where vehicular encroachment is permissible, a sloping curb should be used.

4. **Sidewalk.** Where a sidewalk is present or is to be constructed in an urban area, a curb may be used. Consideration should be given to the type of curb existing or proposed in a similar condition within the adjacent geographical area.

5. **Island.** Where a divisional or directional island is used, it should be raised and corrugated. Section 46-9.0 and the INDOT *Standard Drawings* provide additional information on the design and placement of a raised corrugated island.

6. **Local Practice.** On a State highway, the designer should strive to meet the prevailing local practice where it does not conflict with Department criteria. Where local practice differs, INDOT criteria should prevail. On a non-State facility, local practice will govern.

45-1.05(04)  Design Considerations

The use of a curbed section requires the consideration and implementation of the design considerations as follows.

1. **Drainage.** Department practice limits the allowable amount of water ponding on the roadway. A closed-drainage system is used with a curbed section. The hydraulic analysis will, among other factors, depend on the curb characteristics. These include type of material (concrete or asphalt), cross slope leading up to the curb, and shape of the curb face. It may be necessary
to prevent the gutter flow from overtopping the curb. This will affect the selected curb height. See Chapter 36 for the specific criteria and procedure for drainage analysis.

The minimum profile grade in a curbed section is ±0.3%. Additional consideration should be given to the minimum grade in a curbed superelevation-transition area to avoid drainage problems. The following criteria will alleviate such problems.

a. A minimum profile grade of ±0.5% should be maintained through a superelevation-transition section.

b. A minimum edge of pavement grade of ±0.5% should be maintained through a superelevation-transition section. The equations to be considered for this criterion are as follows:

\[
\begin{align*}
G & \leq -\Delta^* - 0.5 \quad \text{[Equation 45-1.1]} \\
G & \geq -\Delta^* + 0.5 \quad \text{[Equation 45-1.2]} \\
G & \leq \Delta^* - 0.5 \quad \text{[Equation 45-1.3]} \\
G & \geq \Delta^* + 0.5 \quad \text{[Equation 45-1.4]} \\
\Delta^* &= \frac{wne_d}{L_r} \quad \text{[Equation 45-1.5]}
\end{align*}
\]

where,

\[
\begin{align*}
G & = \text{profile grade, \%}; \\
\Delta^* & = \text{effective maximum relative gradient, \%}; \\
w & = \text{width of one traffic lane, m (typically 3.6)}; \\
n & = \text{number of lanes rotated}; \\
e_d & = \text{design superelevation rate, \%}; \\
L_r & = \text{length of superelevation runoff, m}.
\end{align*}
\]

EXAMPLE 45-1.01

To illustrate the combined use of the two criteria, consider the following:

\[
\Delta^* = 0.65\% \text{ in the transition section}
\]

Criterion 1.a. described above excludes a grade between −0.5% and +0.5%.
Criterion 1.b. excludes a grade between -1.15% (via Equation 03-19.1, where \( G \leq -0.65 - 0.5 \), or -1.15), and -0.15% (via Equation 03-19.2, where \( G \geq -0.65 + 0.5 \), or -0.15).

Also, Criterion 1.b. excludes a grade between +0.15% (via Equation 03-19.3, where \( G \leq +0.65 - 0.5 \), or +0.15), and +1.15% (via Equation 03-19.4, where \( G \geq +0.65 + 0.5 \), or +1.15).

Therefore, the profile grade within the transition must be outside the range of –1.15% to +1.15% in order to satisfy both criteria and provide adequate pavement surface drainage.

See the AASHTO *A Policy on Geometric Design of Highways and Streets* for more information.

2. **Cross Slope.** Where an integral curb-and-gutter section is used, the cross slope of the gutter is the same as the adjacent pavement surface. Where a separate curb-and-gutter section is used, the gutter-pan cross slope is as shown in Figure 45-1D.

3. **Roadside Safety.** The placement of a barrier behind a curb must meet placement and height criteria. Chapter 49 discusses roadside-safety criteria relative to a curb.

4. **Future Resurfacing.** The designer should consider the likelihood and depth of a future resurfacing course when determining the initial curb height. For example, the curb height may be determined from the sum of the water-overtopping depth (based on a drainage analysis) and the future resurfacing depth. Because milling of the pavement is becoming more prevalent, additional curb height may not be a consideration.

5. **Parking Considerations.** The curb height next to on-street parking should be 6 in. or less. This will allow clearance for the opening of a car door. The curb height on a street or parking lot with diagonal or perpendicular parking should also be limited to 6 in to prevent underside vehicle damage.

6. **Freeze-Thaw Considerations.** The combined curb-and-gutter design removes the pavement joint away from the face of curb. After several freeze-thaw cycles, a standard curb type may become uneven and present an unsightly appearance; therefore, an integral or combined curb is preferable.

7. **Accessibility.** A curb ramp must be provided at each pedestrian street crossing to provide safe and convenient movement of pedestrians with disabilities. Section 51-1.04 and the INDOT *Standard Drawings* series 604-SWCR provides details on the design and location of a curb ramp.
A sidewalk is considered an integral part of the urban environment. In such an area, a pedestrian frequently chooses to make all or part of his or her trip on foot, and an improved surface is expected. In a rural area a sidewalk is less common, but it may have sufficient value in a developed rural area to merit its construction. This is especially true in the vicinity of a school.

New and reconstructed sidewalks must be in accordance with the Public Right of Way Accessibility Guidelines (PROWAG). See Section 51-1.03 for accessibility requirements.

45-1.06(01) Guidelines for Including Sidewalk within the Project Scope

1. **Sidewalk Currently Exists.** Where a sidewalk currently exists and will be disturbed by construction, the sidewalk should be reconstructed. If a bridge with an existing pedestrian sidewalk is reconstructed, the sidewalk should be retained.

   If a sidewalk exists only on one side of a State highway or bridge, the project will often include the construction of a new sidewalk on the other side. However, the funding and maintenance arrangements will be according to the criteria in Item 5 below.

2. **Sidewalk Does Not Currently Exist on Roadway.** The warrant for a sidewalk depends upon the project location being inside or outside city limits. The following provides guidance for each of these situations.

   a. **Project within City Limits.** At the preliminary field check stage, the designer should arrange a meeting between the appropriate district personnel and city officials to make a collective determination on the need for a sidewalk. If the city officials indicate that a sidewalk is needed and request that it be included as part of the project, the project will include the sidewalk. If Federal-aid funds are used, the Department may elect to help pay for the construction of the new sidewalk. However, if Federal-aid funds are not used, this will require a reimbursement agreement between the State and the city in accordance with Item 5 below. If the city officials indicate that the sidewalk is needed but do not want it included as part of the project, the designer should develop the plans so that a graded grassed area is provided for a future sidewalk. The incorporation will be responsible for installing the sidewalk in the future.

   b. **Project outside City Limits (Town or Rural Area).** The Department may install a sidewalk if it deems it necessary. The need for a sidewalk will be determined as required for each project. No numerical warrants are available. The designer should
consider providing a sidewalk along a roadway where pedestrians are present or would be expected to be present if they had a sidewalk available (i.e., a latent demand exists such as evidence of a pathway along a highway).

Once the decision is made to provide a sidewalk along a roadway, the need for a sidewalk on both sides of the roadway will be determined as required for each project.

3. **Sidewalk Does Not Currently Exist on Bridge.** If a bridge is within the limits of a reconstruction (4R) or 3R project and if its bridge deck will be rehabilitated as part of the project, a sidewalk should be provided on the bridge if provided on the approach roadway. If the bridge deck will not be rehabilitated as part of the reconstruction or 3R project, it will rarely be warranted to perform work solely to provide a sidewalk on the bridge unless a sidewalk exists on the approaching roadway.

Bridge deck rehabilitation may be the only work on a 3R project. A sidewalk may be on the approach roadway or the approach roadway may be a candidate for a future sidewalk based on the discussion in Item 2 above. If so, a sidewalk should be included as part of the bridge deck rehabilitation project.

Once the decision is made to provide a sidewalk on a bridge, one will be constructed on each side, unless there is a justification to place a sidewalk on only one side.

4. **Sidewalk Does Not Currently Exist on Underpass.** An underpass may be within the limits of a project. If the approach roadway will have a sidewalk, it will be provided through the underpass, unless this would involve unreasonable costs to relocate the bridge substructure. A bridge-reconstruction project may involve major work on or the replacement of the bridge substructure. If the bridge passes over a roadway, the designer should consider allowing space for the future addition of a sidewalk through the underpass.

Once the decision is made to provide a sidewalk through an underpass, one will normally be constructed along each side of the underpassing roadway, unless there is a justification to place a sidewalk on only one side.
5. **Funding and Maintenance Considerations.** Sidewalk funding and maintenance considerations are dependent upon project location. The following will apply:

a. **City Limits.** For a sidewalk constructed within city limits, the city will be responsible for the costs of constructing the sidewalk unless Federal-aid funds are used. The State may then participate. If totally funded by the city, a reimbursement agreement will be required between the Department and the city prior to the project letting. The State will be responsible for the cost of right-of-way acquisition and grading required specifically for the sidewalk.

b. **Town or Rural Area.** A new sidewalk constructed in a town or rural area outside of city limits may be funded with State or Federal-aid funds. This includes all costs for right-of-way acquisition, grading, and construction.

c. **Bridge.** Regardless of location, the total cost for a sidewalk on a bridge may be funded with State or Federal-aid funds.

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**45-1.06(02) Sidewalk Design Criteria [Rev. Mar. 2016, Sep. 2016]**

A sidewalk within the public right of way must be in accordance with the Public Right of Way Accessibility Guidelines (PROWAG). See Section 51-1.03 for sidewalk accessibility requirements. Designers should also consider the following:

1. **Width.** The sidewalk width is measured exclusive of the curb, i.e. measured from the back face of curb. A typical sidewalk is 5 ft wide with a 5-ft buffer area between the roadway and sidewalk. If there is no buffer area provided, the sidewalk width should be 6 ft to accommodate any appurtenances which may be included in the sidewalk (see Item 4 below). A high pedestrian volume may warrant a greater width in, for example, a commercial area or school zone. The designer may conduct a detailed capacity analysis to determine the sidewalk width. *Highway Capacity Manual* Chapter 13 should be reviewed for this analysis.

2. **Urban Area.** In a central business district, the entire area between a curb and a building is used as a paved sidewalk.

3. **Appurtenance.** The designer should consider the impacts of a roadside appurtenance within the sidewalk, e.g., street furniture, fire hydrant, parking meter, utility pole, signs. These elements reduce the clear width and may interfere with pedestrian activity. If such an
If an appurtenance is placed within the sidewalk, the sidewalk clear width must be at least 4 feet or the sidewalk should be widened accordingly.

4. **Cross Slope.** The maximum cross slope is 2.00%. For design purposes the cross slope should be set at 1.5% to reduce the likelihood of the maximum being exceeded during construction.

5. **Buffer Area.** If the available right of way is sufficient, a buffer area between the curb and sidewalk is desirable. This area provides space for snow storage and allows for a greater separation between vehicle and pedestrian. The buffer area should be at least 5 ft wide to be effective and should desirably be wider. Although occasionally unavoidable, placing roadside appurtenances within the buffer area is undesirable. The proximity to the traveled way increases the likelihood of a vehicle/fixed-object crash.

6. **Sidewalk on Bridge.** The sidewalk width is measured from the back of an assumed 6” curb. See section 402-6.02(01). Section 404-4.02(03) provides criteria for the selection and location of bridge and pedestrian railing where a sidewalk is present. Section 49-9.02 provides information for shielding the end of a bridge railing.

### 45-2.0 MEDIAN

A median is desirable on a highway with 4 or more lanes. The principal functions of a median are as follows:

1. to provide separation from opposing traffic,
2. to prevent undesirable turning movements,
3. to provide an area for deceleration and storage of left-turning vehicles,
4. to provide an area for storage of a vehicle crossing the mainline at an intersection,
5. to facilitate drainage collection,
6. to provide an area for snow storage,
7. to provide an open green space,
8. to provide a recovery area for an out-of-control vehicle,
9. to provide a refuge area in case of emergency,
10. to minimize headlight glare,
11. to provide an area for pedestrian refuge, or
12. to provide space for future lanes.
45-2.01 Median Width [Rev. April 2016]

The median should be as wide as can be used advantageously. The median width is measured from the edges of the two inside travel lanes and includes the left shoulders or curb offsets. The design width will depend on the functional classification of the highway, type of median, availability of right of way, construction costs, maintenance considerations, acceptable median slopes, the anticipated ultimate development of the facility, operations at crossing intersections, and field conditions. The designer should consider the following to determine an appropriate median width.

1. **Left Turn.** The need for a left-turn bay should be considered in selecting a median width.

2. **Crossing Vehicle.** A median should be approximately 25 ft wide to safely allow a crossing passenger vehicle to stop between the two roadways. Where trucks are commonly present (e.g., truck stop), the median width should be increased to allow a truck to stop between roadways. The appropriate design vehicle for determining the median width should be chosen based on the actual or anticipated vehicle mix of crossroad or other traffic crossing the median.

3. **Signalization.** At a signalized intersection, a wide median can lead to inefficient traffic operation and may increase crossing time.

4. **Median Barriers.** A median barrier may be warranted in a narrow median. Therefore, the median should be wide enough to eliminate the need for a barrier. See Section 49-6.0 for median barriers.

5. **Operations.** Some vehicular maneuvers at an intersection are partially dependent on the median width. These include a U-turn or a turning maneuver at a median opening. The designer should evaluate the likely maneuvers at an intersection and provide a median width that will accommodate the selected design vehicle. See Section 46-8.01 and Item 2 above.

6. **Separation.** From a driver’s perspective, a median width of 40 ft physically and psychologically separates him or her from the opposing traffic.

7. **Uniformity.** A uniform median width is desirable. However, a variable-width median may be advantageous where right of way is restricted, at-grade intersections are widely spaced (2500 ft or more), or an independent alignment is practical.

8. **Other Elements.** The widths of other roadway cross-section elements should not be reduced to provide additional median width.
Chapter 53 provides specific numerical criteria for median width on a new-construction or reconstruction project. On an existing highway, retaining of the existing median width will be determined as required for each project.

45-2.02 Median Type

Figure 45-2A illustrates the available median types: flush, flush with concrete median barrier, raised, or depressed. The following provides additional information on median type.

45-2.02(01) Flush Median

A flush median is used on an urban highway or street. A flush median should be slightly crowned to avoid ponding water in the median area. However, a flush median with a concrete median barrier should be depressed to collect water within a closed-drainage system.

The width for a flush median on an urban street ranges from 4 ft to 16 ft. If the median width is 16 ft or less, the designer should consider using a continuous raised corrugated median or a slightly mounded median curb with 1 to 2 in. edge height. A corrugated type of median should be used where there is little or no anticipation that a motorist will drive onto the median to make a left turn. The INDOT Standard Drawings provide additional details for a corrugated or mounded median. To accommodate a left-turn lane, a flush median should be 14 ft wide. This will allow a 12-ft turn lane and a minimum 2-ft separation between a left-turning vehicle and the opposing traffic.

A two-way left-turn lane (TWLTL) is also considered a flush median. The roadway cross section with a flush median will allow ultimate development for a TWLTL. The figures in Chapters 53, 55, or 56 provide the criteria for TWLTL width. Section 46-5.02 provides information on design details for a TWLTL at an intersection.

A flush median with a concrete median barrier may be used on an urban freeway where the right of way does not allow for the use of a depressed median. For a new-construction or complete reconstruction project, the minimum width of a flush median for an urban freeway is 25 ft. This allows the use of two 12-ft left shoulders and the width of the concrete median barrier. On a partial-reconstruction project, the minimum width may be the existing median width.
45-2.02(02) Raised Median

A raised median is used on an urban highway or street to control access and left turns, and to improve the capacity of the facility. Figure 45-2A illustrates a raised median.

If compared to a flush median, a raised median offer the advantages as follows:

1. mid-block left turns are controlled;
2. left-turn channelization can be more effectively delineated if the median is wide enough;
3. a distinct location is available for traffic signs, signals, pedestrian refuge, or snow storage;
4. the median edges are more discernible during and after a snowfall;
5. drainage collection may be improved; and
6. limited physical separation is available.

If compared to a flush median, the disadvantages of a raised median are as follows:

1. it are more expensive to construct and more difficult to maintain;
2. it may need a greater width to serve the same function (e.g., left-turn lane at an intersection) because of the raised island and offset between curb and travel lane.
3. adverse vehicular behavior may result upon impact of a curb;
4. prohibiting mid-block left turns may overload a street intersection and may increase the number of U-turns;
5. it may complicate the drainage design; and
6. access for an emergency vehicle is restricted.
If a raised median will be used, the designer should consider the following in the design of the median:

1. **Design Speed.** Because of the possible adverse effect that a curb can have on a vehicular behavior if impacted, a raised median should only be used where the design speed is 45 mph or lower.

2. **Curb Type.** Either a vertical or sloping curb with an edge height of 1 to 2 in. or more may be used.

3. **Appurtenance.** If practical, the placement of an appurtenance within the median is discouraged (e.g., traffic signal pole, light standard).

4. **Desirable Width.** If practical, the width should be sufficient to allow for the development of a channelized left-turn lane. This yields an 18-ft median width, assuming the following:
   
   a. a 12-ft turn lane,
   b. a 2-ft curb offset between the opposing through lane and raised island, and
   c. a minimum 4-ft raised island.

5. **Minimum Width.** The minimum width should be 8 ft. This assumes a minimum 4-ft raised island with 2-ft curb offsets on each side adjacent to the through travel lanes. In a restricted location, a continuous vertical curb may be offset 1 ft, and sloping curb may be offset 0 ft. Under this condition, the minimum raised-median width with a vertical curb is 6 ft, and that with a sloping curb is 4 ft.

6. **Raised Island (Paved).** For a raised island up to 16 ft wide, the island should be paved to reduce the maintenance requirements of the median.

7. **Raised Island (Landscaped).** For a raised island of 16 ft or wider, the area between the curbs is backfilled and landscaped. However, where there are numerous signs, bridge piers, etc., in the island, it may be more economical to pave the raised island to eliminate excessive hand mowing.

### 45-2.02(03) Depressed Median [Rev. April 2016]

A depressed median is used where practical on a freeway or other divided rural arterial. A depressed median has better drainage and snow storage characteristics and, therefore, is preferred for a major highway. It provides the driver with a greater sense of comfort and freedom of operation. In the design of a depressed median, the designer should consider the following.
1. **Width.** It should be as wide as practical to allow for the addition of future travel lanes on the inside while maintaining a sufficient median width. See Chapters 53 and 54.

2. **Longitudinal Gradient.** The minimum center longitudinal grade with an unpaved ditch should be 0.5%, or, with a paved ditch 0.3%. Under a restricted condition, a minimum grade of 0.3% or 0.2%, respectively, may be used.

3. **Side Slope.** The side slopes should be 6:1 or flatter. See Section 49-6.04 for slope considerations in front of median barrier.

4. **Ditch.** On new construction, a 4-ft flat-bottom ditch in the center should be considered.

5. **Drainage Inlet.** A drainage inlet should be designed with the top of the inlet flush with the ground or with traversable safety grates on the culvert ends. See Section 49-3.0 for more information.

### 45-3.0 ROADSIDE ELEMENTS

#### 45-3.01 Fill Slope

A fill slope is the slope extending outward and downward from the edge of the shoulder to intersect the natural ground line. The slope criterion depend upon the functional classification, fill height, urban or rural location, project scope of work, and the presence of curbs. For new construction, a 6:1 slope should be used to the edge of the clear zone and, if the slope has not intersected the natural ground line at this point, a 3:1 or flatter slope is used to the toe. Figures 45-3A and 45-3B provide the fill-slope criteria.

Although Figures 45-3A and 45-3B provide specific criteria for a fill slope, consideration must be given to right-of-way restrictions, utility considerations, and roadside development in determining the appropriate fill slope for the site conditions. If practical, a flatter fill slope than indicated should be used.
** PRACTICE POINTER **

Grading for guardrail end treatment should not be shown on the Typical Cross Section sheet.

45-3.02 Cut Slope

45-3.02(01) Slope Rate

On a facility without curbs, a roadside ditch is provided in a cut slope to control drainage. Figure 45-3C provides the criteria for a cut section without a curb. As indicated in Figure 45-3C, the ditch section includes the foreslope, ditch width, and backslope as appropriate for the facility type. On a facility with curbs, a shelf is provided with a backslope beyond the shelf. Where a sidewalk is present or anticipated in the future, a minimum shelf width of 11 ft should be provided. This provides a 1-ft appurtenance strip behind the sidewalk. The minimum shelf width without a sidewalk or anticipated sidewalk may be 5 ft. Applicable criteria are provided in Figure 45-3D. For a section with a curb, sidewalk and ditch, the designer should refer to Figure 45-3C for appropriate criteria beyond the sidewalk. The following provides additional information for an earth or rock cut.

45-3.02(02) Rock Cut (Backslope)

The backslope for a rock cut should not exceed 1:6. For a large rock cut, benching of the backslope may be required. Section 18-2.08 provides the benching criteria for a rock cut.

45-3.02(03) Material and Soils Conditions

The designer must ensure that permanent erosion control is considered in the design of a ditch in a cut slope. The Office of Materials Management may review the existing soils conditions to determine if additional measures may be required to control erosion (e.g., additional topsoil, special plantings, paving). It will be the designer’s responsibility to consider such recommendations for incorporation into the plans. A longitudinal-ditch slope of 1% or steeper will require sodding. A slope of 3% or steeper will require a paved or riprap lining. See Section 17-4.07 for estimating roadside ditch quantities. For more information on the design of ditch lining, the designer should review Part 2, Hydrology and Hydraulics.

45-3.02(04) Roadside Safety
To safely accommodate a run-off-the-road vehicle, the ditch slopes should be as flat as practical. Section 49-3.02 provides specific criteria to determine desirable foreslope and backslope combinations. All hazards within the clear zone are to be removed, relocated, made breakaway, or shielded. See Chapter 49.

45-3.02(05) Hydraulic Design

Part 2, Hydrology and Hydraulics discusses the hydraulic design of a roadside ditch. The depth of the ditch should ensure that the flow line for the design discharge (e.g., Q10) will be below the subgrade intercept with the foreslope. The flattest longitudinal grade for an unpaved ditch should be 0.5%. A flatter longitudinal grade of 0.3% may be used under a restricted condition.

45-3.02(06) Standard and Special Roadside Ditches

For plan development, a roadside ditch can be described as either a standard or special ditch. A standard ditch follows the profile of the roadway. The ditch grade need not be shown on the Plan and Profile sheet. A special ditch does not follow the profile of the roadway. For a special ditch, a separate ditch grade should be defined and shown in the profile view of the Plan and Profile sheet. The cross section of standard and special ditches should be detailed as part of the Typical Section sheet.

45-3.03 Reducing the Use of a 2:1 Slope

A slope of 2:1 or steeper should be avoided on an INDOT project unless it is absolutely necessary. Such a slope is extremely difficult to maintain, is susceptible to erosion problems, and in some soil types has serious slope-stability problems. The use of a 2:1 or steeper slope on a local-public-agency project will be at the discretion of the local public agency.

The acceptability of using a steeper-than-desirable sideslope differs depending on the project design criteria as follows.

45-3.03(01) New-Construction or Reconstruction Project (4R)

On a 4R project which requires additional right of way, the use of a 2:1 slope should be avoided wherever possible. In a deep cut or high fill, the additional right-of-way cost to construct a 3:1 slope beyond the clear zone is a minor consideration. If a 2:1 slope appears to be necessary at a select location, early geotechnical investigation should be conducted to determine its suitability.
In an urban area with limited or costly right of way, a 2:1 slope is permissible. An alternative such as burying a pipe in a ditch to reduce the slope or constructing a mechanically-steepened slope should be evaluated if one of these practices will result in better slope stability. Another alternative is described in Section 36-6.08, which recommends the use of a curb under guardrail along the shoulder at the top of a steep slope with high erosion potential. Details of this practice are shown in the INDOT Standard Drawings.

For an Interstate 4R rehabilitation project, it may not be feasible to upgrade each slope to provide the required clear zone due to environmental constraints or right-of-way limitations. If a slope steeper than 3:1 is retained, it should be evaluated to determine if guardrail is warranted using the figures shown in Section 49-4.04. A slope may also be evaluated using the software included in the AASHTO Roadside Design Guide. See Section 54-4.0.

The designer must prepare Design Exception Level Two documentation where a 2:1 sideslope is proposed on a 4R project. This should be completed at the grade-review stage. The documentation must include a discussion of the economic or environmental reasons for needing a sideslope of 2:1 or steeper.

A slope of 3:1 instead of 2:1 should be used in a rock-cut area. Most rock is sandstone or shale and will not stand vertically. A backslope of 2:1 should be used only where good slope stability or sound rock has been verified.

45-3.03(02) 3R Project

The use of a 3:1 slope should be considered as described in Section 55-4.05(9) Item 2.a. If a steeper slope is required, a 2.5:1 slope should be considered before implementing a 2:1 slope. A slope behind guardrail at a corner of a bridge should not be steepened to 2:1, though the slope may be completely protected by the guardrail.

A location or situation that may warrant a slope of 2:1 or steeper is as follows:

1. roadway widening that encroaches into a wetland;

2. area with restrictive or costly right of way; or

3. slope at the end of a large culvert, bridge spillslope, or other location where it is desirable to protect the slope with riprap.

Where a 2:1 slope is specified, it should be protected with erosion-control blankets and capping soils.
suitable for growing vegetation. The designer should contact the Production Management Division’s landscape architect concerning the possibility of capping a cut or fill slope steeper than 3:1.

45-4.0 BRIDGE OR UNDERPASS CROSS SECTION

The highway cross section must be carried over or under a bridge, which often requires special considerations because of the confining nature of a bridge and its high unit costs. The bridge or underpass section will depend upon the cross section of the approaching roadway, the highway functional classification, and the project scope of work.

45-4.01 Bridge

The road-design criteria will determine the proper cross section width of the roadway, and the bridge design will accommodate the paved approach width across each structure within the project limits. This will provide full continuity of the roadway section for the entire project. This process will, of course, require proper communication between the road designer and bridge designer to identify and resolve any problems.

The bridge cross section will be determined by the project scope of work. For new construction or a bridge project within the limits of a 4R road project, the criteria provided in Chapter 53 will determine the cross section of the bridge. For a bridge project within the limits of a 3R road project, the cross section will be determined from the criteria shown in Chapter 54 or 55. Section 40-6.0 provides project scope-of-work definitions and a map of the State highway system with designated 3R and 4R routes. The following will apply to the cross section of a bridge.

1. **Clear-Roadway Width.** Chapter 53 provides criteria for a new-construction project and a bridge within the limits of a 4R road project. Chapters 54 and 55 provide criteria for a bridge within the limits of a 3R road project on a freeway or non-freeway. For a summary of bridge-width criteria, see Section 59-1.01.

2. **Travelway-Width Reduction.** Upon approaching a narrow bridge, the roadway width must be reduced to allow it to be accommodated by the bridge. The travelway-reduction transition should be designed using the taper rate shown in Figure 502-2J.

3. **Auxiliary Lane.** To determine the additional width needed for an auxiliary lane, the following will apply.
a. Chapter 48 discusses the warrants for and design of an auxiliary lane within an interchange. This may be needed across a bridge, for example, to accommodate vehicular weaving within a full-cloverleaf interchange.

b. Chapter 46 discusses warrants for and the design of an auxiliary lane at an intersection, including two-way left-turn lane, turning roadway, and exclusive turn lane. This may impact the design width of a structure near an intersection.

c. Section 44-2.0 discusses the warrants for and design of a climbing lane. The full width of this lane including shoulders will be provided across a structure.

d. Chapters 53, 54, and 55 provide the width of an auxiliary lane for various project scopes of work (e.g., 3R, 4R) and facility type (e.g., arterial).

4. Cross Slope. On a tangent section, a new or reconstructed bridge will be constructed with a cross slope of 2% sloping away from the crown. The 2% applies to the entire width from the crown to the front face of railing or curb. The crown across the bridge will be in the same location as the approaching roadway’s crown. An existing bridge to remain in place may retain an existing cross slope of 1.5%.

On a superelevated roadway section, a break may be provided between the traveled way and the high-side shoulder. However, on a superelevated bridge, a constant slope at the superelevation rate is provided across the entire curb-to-curb or railing-to-railing width of the bridge. This applies to a fully-superelevated section, or a section within a superelevation transition.

The approach roadway will include a shoulder with a cross slope different from that on the bridge. For example, the typical roadway shoulder cross slope on tangent is 4%. It will be necessary to transition the roadway shoulder slope to the bridge deck slope before reaching the bridge deck. The rate of transition should be consistent with the relative longitudinal slope used for a superelevation transition. This is described in Section 43-3.0.

See Section 59-1.0 for the cross section for a bridge.

5. Median. Section 45-2.0 discusses the design of a median. Twin parallel structures will be used to carry a median across an overpass. For a long span with a sufficiently narrow median, some economy in substructure costs may be realized by constructing a single structure. Depending on site conditions, a single structure may be more cost effective than twin structures where the median width is approximately 30 ft or less on a freeway or 20 ft or less
on another type of road. The median width at an overpass will match the median width on the approach.

6. **Sidewalk.** Section 45-1.06 provides the sidewalk warrants on a bridge. For design of a sidewalk on a bridge, see Chapter 61.

7. **Side Slope.** Section 45-3.0 provides criteria for fill and cut slopes along the roadway. If it is necessary to transition a slope, the transition should be made such that the maximum longitudinal slope along the roadside does not exceed 20:1 at a line measured a distance of 25 ft from the edge of traveled way.

8. **Ramp.** For a bridge on an interchange ramp, the full paved width of the ramp should be provided across the bridge. See Section 48-5.0 for criteria on ramp width.

### 45-4.02 Underpass

The cross section of an underpass has a significant impact on the size of the overpassing structure. The underpass should be designed as described below.

1. **Roadway Section.** The full approach-roadway section, including the median width, should be provided through the underpass section.

2. **Clear Zone.** The roadside clear zone applicable to the approaching roadway section will be provided through the underpass. Section 49-2.0 provides clear-zone criteria, which are a function of design speed, traffic volume, highway alignment, and side slopes. If an auxiliary lane is provided through the underpass, this impacts the clear-zone determination. Section 49-2.0 discusses the width of a clear zone where an auxiliary lane is present.

3. **Travelway-Width Reduction.** Upon approaching a narrow underpass, the roadway width should be reduced to allow the roadway to pass under the bridge. The travelway-reduction transition should be designed using the taper rate shown in Figure 502-2J.

4. **Sidewalk.** Section 45-1.06 provides the sidewalk warrants through an underpass.

5. **Side Slope.** Section 45-4.01 discusses the rate of transition for modifying the rate of a fill or cut slope near an underpass.

6. **Future Expansion.** In determining the cross-section width of a highway underpass, the road designer should also consider the likelihood of future roadway widening. Widening an
existing underpass in the future can be expensive. Therefore, the designer should evaluate the potential for further development in the vicinity of the underpass which would significantly increase traffic volume. If appropriate, a reasonable allowance for future widening may be made to provide sufficient lateral clearance for additional lanes.

7. **Ramp.** For an underpass on an interchange ramp, the full paved width of the ramp including shoulders and the clear-zone width should be provided through the underpass. See Section 48-5.0 for criteria on ramp width.

### 45-5.0 CHANGE IN ROADWAY CROSS SECTION

The transition from a divided facility to one of only 2 lanes is a complex decision-making area for a driver, who may not be expecting the lanes reduction. Therefore, the designer should use the safest criteria practical, whether the transition is permanent or temporary.

The horizontal alignment for a permanent or temporary transition should follow the criteria described in Chapter 43. A temporary connection should be designed as a new facility. This includes, but is not limited to, superelevation, transition length, reverse curves, or the tangent length between curves.

Decision sight distance should be provided to and throughout the transition area. To achieve this objective, the project termini may need to be adjusted.

The following figures illustrate transition design.

1. Figure 45-5A provides the details for a transition from a 2-lane to a 4-lane facility on a curve. The transition may also be designed on a tangent. The designer must consider the design of the horizontal-alignment features. See Chapter 43.

2. Figure 45-5B provides the details for a split transition from a 4-lane to a 2-lane facility on a tangent section.

3. Figure 45-5C provides the details for a split transition from a 4-lane undivided to a 4-lane divided facility on a tangent section.

4. Figure 45-5D provides the details for a split transition from a 4-lane undivided to a 5-lane TWLTL facility on a tangent section.
45-6.0 RIGHT OF WAY

45-6.01 Definitions

The following right-of-way definitions will apply:

1. **Permanent Right of Way.** Right of way acquired for permanent ownership by the State for an activity which is the responsibility of the State for an indefinite period of time. The State obtains the title to the property. Permanent right of way is acquired for roadway, utility accommodation, fill or cut slopes, etc.

2. **Temporary Right of Way.** Right of way required for the legal right of usage by the State to serve a specific purpose for a limited period of time. The period of time is that until a project is completed, for building removal until the building is removed, or for condemnation until three years beyond December of the anticipated letting year at the time of condemnation. Once the activity is completed, the State yields its legal right of usage and returns the land to its original condition as close as practical.

3. **Right-of-Way Easement.** Right of way required with the perpetual right to construct and maintain a public highway and incidental facilities over and across the surface of land. This includes the following:
   a. highway easement (e.g., relocating, cleaning, or repairing a legal ditch);
   b. utility easement for a private facility (e.g., pipeline, private access road); or
   c. storm-sewer easement.

4. **Perpetual Easement.** Right of way acquired with the perpetual or permanent right to construct and maintain an off-road facility such as a sewer line, drainage ditch, or other item (except that under the jurisdiction and control of a county drainage board) outside the highway or service-area right of way.
45-6.02 Width

The minimum right-of-way width will be the sum of the widths of travel lanes, shoulders, median (if applicable), ditches, plus that necessary for fill or cut slopes or for roadside clear zones, whichever is greater. The overall right-of-way width should be increased to provide additional width for the following.

1. **Maintenance.** A 6-ft to 15-ft maintenance area should be provided along each side of the roadway to accommodate maintenance equipment at the top or bottom of a cut or fill slope.

2. **Utility Corridor.** A utility corridor, for an underground or overhead utility, should be provided beyond the roadside clear zone. Chapter 104 provides additional information on the placement of utility lines within the highway right of way.

3. **Future Expansion.** The designer should initially consider obtaining sufficient right of way to meet the anticipated long-term corridor growth. This may include obtaining additional right of way for the following:
   
   a. a wider median to allow for the addition of future through travel lanes;
   
   b. expansion of an existing interchange;
   
   c. a future interchange; or
   
   d. expanding an existing 2-lane facility to a 4-lane divided highway.

The right-of-way width should be uniform, but this is not a necessity. In an urban area, a variable width may be necessary due to existing development or varying side slopes. Embankment heights may make it desirable to vary the right-of-way width. Right-of-way limits will likely have to be adjusted at each intersection or freeway interchange. Other right-of-way controls to be considered are as follows.

1. At a horizontal curve or intersection, additional right of way may be warranted to ensure that the necessary sight distances are always available in the future.

2. Where the necessary right-of-way width cannot be reasonably obtained, the designer should consider using steeper slopes, revising grades, or using retaining structures.

3. Right-of-way considerations at an interchange are discussed in Chapter 48.

Chapter 85 provides additional criteria for establishing the right-of-way limits. The designer will coordinate with the Office of Real Estate on the purchase of right of way.
45-7.0 FRONTAGE ROAD

45-7.01 General

A frontage road serves numerous functions, depending on the type of facility served and the character of the surrounding area. It may be used to control access to the facility, to function as a street serving adjoining property, or to maintain circulation of traffic on each side of the main highway. A frontage road segregates local traffic from the higher-speed through traffic and serves drives of residences or commercial establishments along the highway. A connection between the main highway and frontage road, usually provided at a crossroad, furnishes access between the through road and adjacent property. Thus, the through character of the highway is preserved and is unaffected by subsequent development along the roadside.

A frontage road may be used with a facility of any functional classification. It greatest use is adjacent to a freeway where its primary function is to distribute and collect traffic between local streets and the freeway interchanges. A frontage road is also desirable along an arterial street in either an urban or suburban area.

Despite its advantages, the use of a continuous frontage road on a relatively high-speed arterial street with intersections at grade may be undesirable. At a cross street, the various through and turning movements greatly increases the accident potential. Multiple intersections are also vulnerable to wrong-way entrances. Traffic operations are improved if the frontage road is located a considerable distance from the main highway at the intersecting crossroad in order to lengthen the spacing between successive intersections along the crossroad. See Section 45-7.03.

A frontage road is parallel to the through roadway. It may or may not be continuous, and it may be provided on one or both sides of the arterial.

For a private frontage or access road, an economic analysis needs to be completed to ensure that the construction of the frontage road will be cost effective versus the purchasing of the property.

45-7.02 Functional Classification

The design elements of pavement width, cross slope, horizontal or vertical alignment, etc., should be provided consistent with the functional operation of the frontage road. The same considerations relative to functional classification, design speed, traffic volume, etc., apply to a frontage road as they would to another highway.
For a high-traffic volume, continuous frontage road, the desirable functional classification will be one level below that of the main highway classification.

For a low-traffic volume, non-continuous frontage road, the design functional classification should be local road or street.

45-7.03 Design

45-7.03(01) Design Elements

The selection of the appropriate design criteria is based on the functional classification of the frontage road. Once the functional classification has been determined, the appropriate design speed, lane and shoulder widths, etc., from the figures in Chapters 53 through 55, can be selected.

45-7.03(02) One-Way Versus Two-Way Operation

From an operational and safety perspective, a one-way frontage road is preferred to two-way. A one-way operation may inconvenience local traffic to some extent, but the advantages in reducing vehicular and pedestrian conflicts at an intersecting street compensates for this inconvenience. There is some savings in pavement and right-of-way width. A two-way frontage road at a high-traffic volume, at-grade intersection complicates crossing and turning movements. An off ramp (e.g., slip ramp) joining a two-way frontage road should not be used because the potential for wrong-way entry is increased.

A two-way frontage road may be considered for a partially-developed urban area where the adjoining street system is so irregular or so disconnected that one-way operation would introduce considerable added travel distance and cause undue inconvenience. A two-way frontage road may also be appropriate for a suburban or rural area where points of access to the through facility from the frontage road are widely spaced.

45-7.03(03) Outer Separation

The area between the main highway and a frontage road is the outer separation. This separation functions as a buffer between the through traffic on the main highway and the local traffic on the frontage road. This separation also provides space for shoulders and ramp connections to or from the through facility.
The wider the outer separation, the less influence local traffic will have on through traffic. A wider separation lends itself to the landscape treatment and enhances the appearance of both the highway and the adjoining property. The outer separation between the through arterial and the frontage road should be 100 ft in a rural area or 60 ft in an urban area. These distances are measured between the edges of the through lanes for the main highway and frontage road. The intersection of the frontage road and crossroad should be 160 ft or more from the intersection of the arterial and crossroad. This lengthens the spacing between successive intersections along the crossroad. The minimum width of outer separation will be that required for the shoulder adjacent to the main highway, the frontage road shoulder or offset, and a median barrier.

A substantial width is particularly advantageous at an intersection with a cross street. A wide outer separation minimizes vehicular and pedestrian conflicts. At an intersection, the outer separation should be based on future traffic considerations.

45-7.03(04) Access

The connection between the main highway and the frontage road are an important design element. On an arterial with slow-moving traffic and a one-way frontage road, a slip ramp or simple opening in a narrow outer separation may work reasonably well. A slip ramp from a one-way frontage road to a freeway is acceptable. However, a slip ramp from a freeway to a two-way frontage road is undesirable as it tends to induce wrong-way entry onto the freeway and may cause crashes at the intersection of the ramp and frontage road. Therefore, on a freeway or other arterial with high operating speeds and a two-way frontage road, the access to the freeway should be provided at an interchange. Details for the ramp and frontage road design are provided in Section 48-6.04.

45-7.04 Design Considerations for Frontage-Road and Local-Road Intersection

Section 40-8.0 discusses the procedure for processing an exception to an INDOT design criterion. These apply to the design of a frontage road.

An existing, reconstructed, or proposed intersection between a frontage road and another facility may need to include a relatively restricted horizontal or vertical alignment on the frontage road as it approaches the intersection. Reduced alignment features near an intersection may be used, assuming that a prudent driver will reduce speed as the vehicle approaches the intersection with the higher-volume facility. Therefore, reduced alignment features for that portion of the frontage road near the intersection may be incorporated if most of the following conditions are met.

1. The frontage road is in a rural area.
2. The road has the appearance of a frontage road.

3. The frontage road does not have a length of over 2500 ft of open-highway conditions that could lead a motorist to conclude that he or she is on a through road.

4. The design speed of the frontage road is 50 mph or lower.

5. There is a sufficient tangent length of 500 to 650 ft to allow for placement of advance curve warning and intersection signs.

6. The projected AADT on the frontage road must be 750 or less.

7. The intersection approach should be controlled with a stop sign for the foreseeable future.

8. Stopping sight distance for the design speed on the frontage road or the local road is available at the approach.

Failure to be in accordance with one of these criteria should not preclude submitting a design exception request for reduced alignment features if a valid justification can be presented. Such factors as heavy development along the road; a posted speed limit lower than the design speed; adverse impacts to property owners and the environment; stable, but higher than recommended, AADT; construction costs; adequate advance signing; and predicted driver reaction to the highway alignment should be considered.
1. Widening as required to provide barrier clearance and lateral support
2. Barrier offset varies from 0.0 ft to 2 ft desirable
3. 0.0 ft minimum, 2 ft desirable

Usable Shoulder Width

Figure 45-1A
<table>
<thead>
<tr>
<th>Paved Shld. Width, (ft)</th>
<th>Shoulder Cross Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 4</td>
<td>2% $^1$</td>
</tr>
<tr>
<td>&gt; 4</td>
<td>2% $^1$ for the 2 ft closest to the travel lane, then 4%</td>
</tr>
</tbody>
</table>

Notes:

1. Where the travel lane tangent cross slope differs from 2%, the shoulder cross slope should match the travel lane cross slope.
2. The shoulder pavement section should be as described in Section 52-9.02(06).

PAVED-SHOULDER CROSS SLOPES, TANGENT SECTION, WITH UNDERDRAINS

Figure 45-1A(1)
<table>
<thead>
<tr>
<th>Paved Shld. Width, (ft)</th>
<th>Shoulder Cross Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 4</td>
<td>2% ¹</td>
</tr>
<tr>
<td>&gt; 4</td>
<td>4%</td>
</tr>
</tbody>
</table>

Notes:

¹ Where the travel lane tangent cross slope differs from 2%, the shoulder cross slope should match the travel lane cross slope.

2. The shoulder pavement section should be as described in Section 52-9.02(06).

**PAVED-SHOULDER CROSS SLOPES, TANGENT SECTION, WITHOUT UNDERDRAINS**

*Figure 45-1A(2)*
PARALLEL CURB

\[ N = \frac{L}{22 \ (TQ \ 26)} \]

\[ \theta = 30^\circ \]

\[ L \]

\[ 18 \]

\[ 16 \]

\[ 16 \]

\[ 16 \]

\[ 2.5 \]

\[ A = 17 \]

\[ B = 10 \]

\[ N = \frac{L - 2.5}{18} \]

\[ \theta = 45^\circ \]

\[ L \]

\[ 13 \]

\[ 13 \]

\[ 13 \]

\[ 13 \]

\[ 6.5 \]

\[ A = 18.5 \]

\[ B = 12 \]

\[ N = \frac{L - 6.5}{13} \]

Note: All linear dimensions in feet.

CURB PARKING CONFIGURATIONS

Figure 45-1B
Key:

L = given curb length with parking spaces

N = number of parking spaces over distance L

A = required distance between face of curb and back of stall, assuming that bumper of parked car does not extend beyond curb face.

B = desirable clear distance needed for a parked vehicle to back out of stall while just clearing adjacent parked vehicles.

All dimensions are in feet unless otherwise noted.

* See Tables in Chapters Fifty-three and Fifty-five for parking lane widths.

CURB PARKING CONFIGURATIONS
(Continued)

Figure 45-1B
<table>
<thead>
<tr>
<th>PARKING CONDITION</th>
<th>WIDTH (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No parking on either side of street</td>
<td>32</td>
</tr>
<tr>
<td>Parallel parking on one side</td>
<td>32</td>
</tr>
<tr>
<td>Parallel parking on both sides</td>
<td>40</td>
</tr>
<tr>
<td>Angle parking on one side</td>
<td>52</td>
</tr>
<tr>
<td>Angle parking on one side, parallel on the other</td>
<td>60</td>
</tr>
<tr>
<td>Angle parking on both sides</td>
<td>82</td>
</tr>
<tr>
<td>Parallel parking on both sides, with lane lines</td>
<td>64</td>
</tr>
<tr>
<td>Angle parking on both sides, with lane lines</td>
<td>106</td>
</tr>
</tbody>
</table>

**DESIRABLE STREET WIDTH WITH ON-STREET PARKING**

*Figure 45-1C*
CURBING TYPES

Figure 45-1D
MEDIAN WIDTH DEFINITIONS

Figure 45-2A
### FACILITY | FILL HEIGHT | FILL SLOPE (1)
--- | --- | ---
Freeways, Urban/Rural Arterials, Urban/Rural Collectors | | 6:1 to clear zone edge; 3:1 maximum to toe. See Section 49-2.03
Rural Local Roads | 0-30 ft > 30 ft | Desireable 4:1 Maximum 3:1
Urban Local Streets | All | 3:1 Maximum

(1) Slopes shown in table are for new construction only. See Chapter Forty-nine for reconstruction.

**TYPICAL FILL SLOPES**
(Non-Curbed Facilities)

Figure 45-3A
<table>
<thead>
<tr>
<th>FACILITY</th>
<th>X</th>
<th>FILL HEIGHT</th>
<th>FILL SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeways</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Rural Arterials Rural Collectors</td>
<td>5 ft (1)</td>
<td>All</td>
<td>6:1 to clear zone edge: 3:1 or flatter to use</td>
</tr>
<tr>
<td>Rural Local Roads</td>
<td>5 ft (1)</td>
<td>0-30 ft</td>
<td>Des: 4:1 Max.: 3:1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;30 ft</td>
<td>3:1 Maximum</td>
</tr>
<tr>
<td>Urban Arterials Urban Collectors</td>
<td>11 ft (2)</td>
<td>All</td>
<td>3:1 Maximum</td>
</tr>
<tr>
<td>Urban Local Streets</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) If sidewalks are present or anticipated, this dimension should be 11 ft.
(2) If sidewalks are not present or anticipated, this dimension may be reduced to 5 ft.

**TYPICAL FILL SLOPE**
(Curbed Facilities)

**Figure 45-3B**
<table>
<thead>
<tr>
<th>FACILITY</th>
<th>FORESLOPE</th>
<th>DITCH WIDTH (2)</th>
<th>BACKSLOPE (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway</td>
<td>6:1</td>
<td>1.2</td>
<td>4:1 for 20 ft</td>
</tr>
<tr>
<td>Arterial or Rural Collector</td>
<td>6:1</td>
<td>1.2</td>
<td>4:1 for 20 ft</td>
</tr>
<tr>
<td>Urban Collector</td>
<td>Des. 6:1</td>
<td>1.2</td>
<td>4:1 for 4 ft</td>
</tr>
<tr>
<td>Rural Local Road</td>
<td>V ≥ 50 mph</td>
<td>4:1 (max)</td>
<td>4:1 (max)</td>
</tr>
<tr>
<td>V ≤ 45 mph</td>
<td>3:1 (max)</td>
<td>Des. 4 ft</td>
<td>3:1 (max)</td>
</tr>
<tr>
<td>Urban Local Street</td>
<td>3:1 (max)</td>
<td>Des. 4 ft Min. “V”</td>
<td>3:1 (max)</td>
</tr>
</tbody>
</table>

**Notes:**

1. See Sections 49-2.0 and 49-3.0 to determine lateral extent of the foreslope in a ditch section.
2. For a rock cut, see Section 45-8.0. Figure value may be exceeded where drainage capacity or other considerations warrant.
3. Value is for earth cut and represents maximum slope. See Section 45-8.0 for typical rock-cut sections.

**TYPICAL CUT SLOPE**
(Non-Curbed Facility)

**Figure 45-3C**
*50:1 where sidewalk exists, is to be constructed or is anticipated to be constructed in the future (may be sloped away from curb)

<table>
<thead>
<tr>
<th>FACILITY</th>
<th>SHELF WIDTH(^{(2)}) (ft)</th>
<th>BACKSLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeways</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Rural Facilities</td>
<td>11 ft(^{(1)})</td>
<td>4:1 Maximum</td>
</tr>
<tr>
<td>Urban Arterials</td>
<td>11 ft(^{(1)})</td>
<td>4:1</td>
</tr>
<tr>
<td>Urban Collectors</td>
<td>11 ft(^{(1)})</td>
<td></td>
</tr>
<tr>
<td>Urban Local Streets</td>
<td>11 ft(^{(1)})</td>
<td>3:1</td>
</tr>
</tbody>
</table>

**Note:**

(1) Includes a 1-ft appurtenance strip behind the sidewalk. If no sidewalk is present or anticipated, this dimension may be reduced to 5 ft.

(2) Drainage facilities may be required between shelf and backslope.

**TYPICAL CUT SLOPES**
(Curved Facility)

**Figure 45-3D**
S = (DESIGN) SUPERELEVATION RATE FOR THE HORIZONTAL CURVE

1. Minimum Radius. The minimum radius is dependent upon the design speed, functional classification and the superelevation rate. See Section 43-2.0 for additional information.

2. Superelevation Transition. The superelevation transition is dependent upon the design speed, radius and superelevation rate. See Section 43-3.0 for additional information.

3. Width Transition. Use the taper rates provided in Figure 76-2B. However, the taper rate should not be less than 40:1.

EXAMPLE OF A CURVED ALIGNMENT TRANSITION
(2-Lane Undivided to a 4-Lane Divided)

Figure 45-5A
Minimum Radius. The minimum radius is dependent upon the design speed, functional classification and the superelevation rate. See Section 43-2.0 for additional information.

Width Transition. Use taper rates provided in Figure 76-2B. However, taper rate should not be less than 40:1.

SPLIT TRANSITION
(2-Lane Undivided to a 4-Lane Divided)

Figure 45-5B
Minimum Radius. The minimum radius is dependent upon the design speed, functional classification and the superelevation rate. See Section 43-2.0 for additional information.

SPLIT TRANSITION
(4-Lane Undivided to a 4-Lane Divided)

Figure 45-5C
TWO-WAY LEFT TURN LANE TRANSITION

Figure 45-5D

Note: See MUTCD, INDOT Standard Drawings and Chapter Seventy-six for proper striping and delineation details.
CHAPTER 50

Economic Analysis

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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<th>Revision Date</th>
<th>Sections Affected</th>
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<td>Apr. 2016</td>
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CHAPTER 50

ECONOMIC ANALYSIS

50-1.0 GENERAL [Rev. Apr. 2016]

The material provided in this Chapter is intended to provide a methodology to evaluate the cost effectiveness of various safety improvement measures at a specific location.

The designer is responsible for ensuring that the design of the project reflects a cost-effective expenditure of the available construction funds. This applies to the design of individual elements (e.g., roadway width, intersection, traffic signal, bridge width, or culverts). The cost-effective evaluation will be based on the judgment and subjective analysis of the designer. A design may sometimes warrant an analytical cost-effective evaluation. This may include, for example, a safety improvement project which will be extremely expensive, or a 3R project which is not in accordance with the criteria shown in Chapter 55. Section 50-2.0 discusses the Department’s cost-effectiveness procedures.

Value engineering is a methodology to review alternatives and to suggest choices that provide a reasonable product without reducing its quality. The Department’s value engineering policy is in accordance with the US Code of Federal Regulations, 23 CFR Part 627. Section 50-3.0 describes INDOT’s value engineering policy and procedures.

50-2.0 COST-EFFECTIVE ANALYSES

50-2.01 General

The criteria in this Manual reflect general cost-effective considerations and are applicable to a wide range of conditions. However, because of the need to develop design criteria for widespread application, they must inherently assume typical benefits and typical costs that would normally be encountered in the selection and design of a project. What is actually encountered for a specific project or site may vary widely in terms of expected benefits and expected costs. It is therefore appropriate to consider the cost-effectiveness of applying the normal design criteria to an individual project or site.

The cost-effective analysis will be conducted by the application of engineering judgment. A rough estimate of construction and right-of-way costs is usually available. The designer has likely evaluated the projected traffic volumes, accident history, and the project impacts on right
of way, the environment, and utility relocation. Once the designer evaluates the likely benefits and costs of the proposed improvement, it is often obvious whether or not a design element under consideration is cost effective. This approach is the most practical in the interest of time. Therefore, engineering judgment will most often be used to conduct the cost-effective analysis.

An analytical cost-effect evaluation may be warranted. The following discusses the basic types of cost-effective methodologies used by INDOT. For additional information on cost-effective methodologies, the user should review NCHRP Synthesis 142 _Methods of Cost-Effectiveness Analysis for Highway Projects._

The users of any cost-effective methodology should recognize its limitations. These include the following.

1. The research data to establish critical relationships (e.g., an accident-reduction factor for flattening a vertical curve) may have questionable validity. The research may have made assumptions which are not universally applicable, or several research studies may have yielded conflicting results. There may be no data available to establish a critical relationship.

2. A cost-effective methodology may require significant amounts of data, and it may require considerable effort to perform.

3. A cost-effect study can only consider those impacts which are quantifiable and which can be assigned a realistic monetary value. It cannot realistically incorporate the impacts of such factors as general design consistency, aesthetics, land values and uses, access, driver convenience and comfort, social ramifications, or environmental consequences.

Therefore, the results of a cost-effective analysis should only serve as a tool to the decision maker. Despite its analytical approach, there is nonetheless a great deal of subjectivity in the analysis. The final decision must place the results in proper perspective when considering the limitations of the cost-effective methodology.

### 50-2.02 User Benefit-Cost Analysis

This approach estimates the total user benefits and costs for a project as a whole or for an individual design element within a project. The methodology considers user benefits such as savings in vehicular operation costs, reduced driving time, and reduced accidents. It considers direct project costs such as preliminary engineering, construction, right of way, and maintenance. The objective is to compare overall benefits to overall costs to determine the economic feasibility
of the proposed project or improvement to a specific design element. The comparison may be made by means of economic techniques including present worth, benefit-cost ratio, rate of return, or payback period.

Many cost-effective methodologies have been developed and many references exist which address user benefit-cost analyses. The standard reference is the AASHTO publication *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements*. The publication’s basic approach can be summarized as follows:

1. **Select Cost Factors.** The *Manual* provides highway user cost data for a base year of 1975. The user of the methodology must select multipliers to convert such data to the year under study.

2. **Select Economic Study Model.** A method to measure the cash outward and inward flows in equivalent dollars by use of a compound interest must be selected. INDOT has selected a discount rate of 4% to calculate present value. An analysis period (e.g., twenty years for new construction) must also be selected (see Section 50-2.03).

3. **Estimate Project Costs.** These include construction, right-of-way, and maintenance costs.

4. **Calculate Unit User Costs.** The user costs, as a function of traffic characteristics and highway geometry, should be estimated for the alternative designs including the do-nothing alternative. User costs include vehicular operating cost, travel time, accident costs, and fares.

5. **Calculate User Benefits.** The benefits for savings in vehicular operating costs, travel time, accident costs, and fares should be estimated.

6. **Convert to Annual User Benefits.** It is necessary to convert all benefits to an annual amount.

7. **Estimate Residual (Salvage) Value.** At the end of a facility’s or design element’s service life, some value will likely remain. This value should be estimated and its worth included in the methodology to offset project costs.

8. **Determine Present Value.** The stream of user benefits and user costs over the design service life must be converted to a present value for comparison between the two.
50-2.03 Safety Benefits Based on Accident History

Accident history is usually the best indicator of future accident experience. Therefore, if the data is available and if valid, it is possible to calculate with some precision the cost-effectiveness of a proposed highway safety countermeasure. This approach is applicable to any assessment of the safety cost-effectiveness of a design element intended to reduce the frequency and severity of accidents, assuming that the pertinent information is available. Because accident history can only be obtained for an existing facility, the procedures described below are only used for a safety-improvement project or a 3R safety enhancement. Section 55-8.0 provides a discussion on how to analyze the accident data.

The controlling factor in this analysis is the benefit to cost ratio (B/C). If the B/C ratio is less than 1, the proposed improvement is not economically prudent. If the B/C ratio is 1 or greater, the improvement is economically prudent. If the B/C ratio is less than but very close to 1, the secondary benefits resulting from the proposed improvement should be analyzed before abandoning the proposed improvement.

The following provides INDOT’s procedure for evaluating the safety benefits of a project improvement based on accident history.

50-2.03(01) Definitions

1. **Equivalent Uniform Annual Benefit (EUAB)**. The projected annual dollar savings amortized over the service life of the improvement. This savings is based on accident reduction and other related cost savings.

2. **Equivalent Uniform Annual Cost (EUAC)**. The projected annual cost amortized over the service life of the improvement. This cost is based on the initial cost, annual maintenance cost, and the terminal (salvage) value of the improvement.

3. **Net Annual Benefit (NAB)**. The difference between the equivalent uniform annual benefit and the equivalent uniform annual cost.

4. **Capital Recovery Factor (CRF)**. The factor used to determine the annual cost with interest to recover the capital investment during the expected service life of the improvement for an equal payment series.

5. **Present-Worth Factor (PWF)**. The factor used to determine the present-day value of the projected economic benefits during the expected service life of the improvement. The
present-worth factor for single payment \((PWF_{SP})\) is used when determining the present-day worth of the terminal value of the improvement. The present-worth factor for equal payment series \((PWF_{EPS})\) is used when determining the present-day value of the annual maintenance costs.

6. **Service Life.** The time period that the improvement can reasonably be expected to impact accident experience. The expected service life should reflect this time period and is not necessarily the physical life of the improvement.

7. **Accident Reduction Factor (ARF).** The expected percent reduction in accidents based on the type of improvement.

8. **Accident Projection Factor (APF).** The factor used to project the number of accidents in a given year. It is assumed to be equal to the factor used to project the increase in AADT. Accidents are assumed to increase at the same rate as the AADT.

**50-2.03(02) Criteria and Constants**

The following criteria and constants should be used in computing the B/C ratio. Any deviation from these criteria or constants should be documented in the project files and, where necessary, an informational copy should be furnished to FHWA. The designer should consider the following:

1. **Accident Costs.** To evaluate a project on the same basis, benefits should be computed with the accident-cost values shown in Figure 50-2A, Accident Cost Per Accident ($).

2. **Service Life.** Figure 50-2B shows service lives of various improvements. Costs and benefits should be based on these time periods.

3. **Interest Rate.** An interest rate of 4% should be used. Figure 50-2C, 4% Interest Factors for Annual Compounding Interest, provides the present-worth and capital-recovery factors for a 4% interest rate.

4. **AADT and Accident Projection.** The designer should assume a 2% increase in AADT and accidents per year over the previous year, unless better data or method of projection is available.

5. **Accident Reduction Benefits.** INDOT is currently using ARFs developed by the State of Missouri. These factors are shown in Section 50-2.03(05); see Figure 50-2G, Missouri Accident Reduction Factors). The ARF should be applied to the total number of accidents,
regardless of the number of people or vehicles involved, when calculating accident reduction benefits. Examples are as follows.

a. For a two-car property-damage-only accident, use the ARF from Figure 50-2G times $3,000, the accident cost from Figure 50-2A, Accident Cost Per Accident ($).

b. For a two-car accident where one car is property-damaged only and two personal injuries occur in the other car, use the ARF from Figure 50-2G times $37,000, the accident cost from Figure 50-2A.

For an improvement that involves multiple alternates, Equation 50-2.1 should be used to calculate the total percent accident reduction for each type of accident.

\[
ARP_t = ARP_1 + \frac{ARP_2(100 - ARP_1)}{100} + ARP_3\left(\frac{100 - ARP_1}{100}\right)\left(\frac{100 - ARP_2}{100}\right) \quad \text{(Equation 50-2.1)}
\]

Where:

\[
ARP_t = \text{total percent accident reduction for multiple improvements}
\]

\[
ARP_1 = \text{the largest percentage reduction in accidents of one of the improvements}
\]

\[
ARP_2 = \text{the second largest percent reduction in accidents of one of the improvements}
\]

\[
ARP_3 = \text{the third largest percentage reduction in accidents of one of the improvements}
\]

For more information on how to determine accident reduction factors, the user should review the Institute of Transportation Engineers publication, Selecting and Making Highway Safety Improvements, a Self Instructional Text TTC-440.

6. **Secondary Benefits.** Secondary benefits, such as improved capacity or other economic benefits, will not be included in the final computed B/C ratio of the selected alternate solution. Secondary benefits may be used in the B/C computational ratios of the alternate improvements studied in determining the selection of the preferred alternate but should not be used for the final B/C ratio.

7. **Equivalent Uniform Annual Benefit (EUAB) and Equivalent Uniform Annual Cost (EUAC).** A summary of the calculations required to determine EUAB, EUAC, and the B/C ratio are shown in Section 50-2.03(03). Example calculations for determining B/C ratios are shown in Section 50-2.03(04).
50-2.03(03) Summary of Steps to Determine the Benefit-Cost Ratio and Net Annual Benefit

The following provides a step-by-step procedure which can be used to compute the B/C ratio and the NAB:

1. Collect accident data and identify accident pattern (see Section 55-8.0).

2. Identify the proposed safety improvement (e.g., flatten horizontal or vertical curve, widen roadway or bridge width, add exclusive left-turn lane, provide traffic signal).

3. Determine the expected service life of the proposed improvement from Figure 50-2B, Service Life.

4. Estimate the construction costs and expected annual maintenance costs.

5. Assuming that the accident data will parallel the AADT, estimate accident reduction for each severity class and for each year of the service life of the improvement as follows:

\[ \text{AR} = (N_a)(ARF)(APF_2) \]  \hspace{1cm} \text{(Equation 50-2.2)}

Where:

- \( \text{AR} \) = Accident reduction by year of service life
- \( N_a \) = Number of accidents (from accident data)
- \( ARF \) = Accident reduction factor (from existing records, judgment, or Figure 50-2G)
- \( APF_2 \) = Accident projection factor

6. Assign values to accident reductions using data from ARF in Figure 50-2G, Missouri Accident Reduction Factors. Compute the accident reduction benefits as follows:

\[ \text{ARB} = (\text{AR})(AC_3) \]  \hspace{1cm} \text{(Equation 50-2.3)}

The result of this step is the gross dollar figure for the total annual benefits for each year of the service life of each improvement.

7. Estimate secondary benefits, wherever possible, and include them in the gross benefit figure but do not include them in the final B/C computation of the selected alternate.
8. Convert gross benefits from Step 6 above to the EUAB as follows:

a. Adjust the benefits to the present-day values by multiplying each year’s total benefit, from Step 6 above, by the present-worth factor for that year from Figure \(50-2C\), 4% Interest Factors for Annual Compounding Interest.

b. Add up all of these adjusted benefits.

c. Multiply the total of the adjusted benefits by the CRF from Figure \(50-2C\) for the last year of the improvement's service life.

d. The formula for the above steps is as follows:

\[
EUAB = \left(CRF\right)\text{(Summation of Yearly-Adjusted Benefits)} \quad \text{(Equation 50-2.4)}
\]

9. Convert the gross costs to the EUAC as follows:

a. Multiply the annual maintenance cost by the present-worth factor for equal payment series for the last year of the improvement’s service life to determine the cumulative maintenance cost.

b. Add the initial cost to the total of the cumulative maintenance costs.

c. Multiply the terminal value by the present-worth factor for single payment for the improvement's last service year and subtract that amount from the result of Step 9.c.

d. Multiply the result of Step 9.d. by the CRF for the improvement's last service year.

e. The formula for the above steps is as follows:

\[
EUAC = CRF\left[I_c + (M_{\text{ac}})\left(PWF_{EPS}\right) - T\left(PWF_{sp}\right)\right] \quad \text{(Equation 50-2.5)}
\]

Where:

- \(CRF\) = Capital recovery factor for the last year of the improvement’s service life
- \(I_c\) = Initial cost
- \(M_{\text{ac}}\) = Annual maintenance cost
- \(PWF\) = Present-worth factor
- \(PWF_{EPS}\) = Present-worth factor (equal-payment series)
10. Calculate the B/C ratio by dividing the EUAB by the EUAC as follows:

\[
B/C = \frac{EUAB}{EUAC} \quad \text{(Equation 50-2.6)}
\]

11. Calculate the NAB by subtracting the EUAC from the EUAB as follows:

\[
NAB = EUAB - EUAC \quad \text{(Equation 50-2.7)}
\]

50-2.03(04) Example Calculations for Benefit-Cost Ratio and Net Annual Benefit

The following are two examples for determining the B/C ratio and the NAB.

*********

Example 50-2.1

Given: S.R. 62, an Urban Collector
Non-freeway 3R Project
Horizontal curve which meets the criteria described in Section 55-4.03, but has a history of accidents as shown in Figure 50-2D, Accident Summary (Example 50-2.1).

Problem: Determine if realignment of the horizontal curve will be cost effective

Solution: The following steps from Section 50-2.03(03) apply:

Step 1: Collect accident data. The accident data is provided in Figure 50-2D.

Step 2: Identify the proposed safety improvement. The selected improvement is to realign the horizontal curve.

Step 3: Determine the service life of improvement. From Figure 50-2B, Service Life, the expected service life for a horizontal alignment change is 20 years.

Step 4: Estimate initial construction and annual maintenance costs. From similar projects, the construction cost is estimated to be $750,000 with annual maintenance after
realigniment to be $3,000. After 20 years, the terminal (salvage) value is expected to be $20,000.

Step 5:

Estimate the assumed accident reduction for each accident type and for each year of service life. The following will apply.

a. From Figure 50-2G, the ARF is 50%.

b. The ARF is assumed to be 2% per year; see Section 50-2.03(02) Item 4 and Figure 50-2E, Accident Reduction Benefits (Example 50-2.1), column 2.

c. From Figure 50-2D, the average annual PDO accidents is 5.66 and average annual F/I accidents is 2.33.

d. Using Equation 50-2.2, Figure 50-2E, columns 3 and 4 show the expected number of PDO and F/I accidents to be reduced.

Step 6:

Compute accident reduction benefits. The following will apply; see Figure 50-2E:

a. Column 5. Determine the benefits of the reduced number of PDO accidents by multiplying the value in column 3 by $3,000, from Figure 50-2A, Accident Cost Per Accident ($), using Equation 50-2.3.

b. Column 6. Determine the benefits of the reduced number of F/I accidents by multiplying the value in column 4 by $37,000, from Figure 50-2A, using Equation 50-2.3.

c. Column 7. Determine total benefit of the reduced number of accidents by adding columns 5 and 6.

d. Column 8. Determine the present-worth factor from Figure 50-2C, 4% Interest Factors for Annual Compounding Interest.

e. Column 9. Determine the present worth of the benefits from the reduced number of accidents by multiplying column 7 by column 8.

f. Total. Determine the total yearly benefits by summing the values in column 9. The total yearly benefit for this realignment example is $846,958.

Step 7:

Estimate the secondary benefits. For this example, there are no secondary benefits.
Step 8: Convert gross benefit from Step 6 to EUAB. The CRF factor from Figure 50-2C for 20 years is 0.0736. Use Equation 50-2.4 to obtain the following:

\[ EUAB = 0.0736 \times 846,958 = 62,336 \]

Step 9: Convert gross costs to EUAC. Using Equation 50-2.5:

\[ EUAC = (0.0736) \times [\$750,000 + \$3,000(13.5903) - \$20,000(0.4564)] = 57,529 \]

Where:

- CRF = Capital recovery factor for the last year of the improvement’s service life = 0.0736 at 20 years (from Figure 50-2C)
- \( I_c \) = Initial cost = $750,000
- \( PWF_{EPS} \) = Present-worth factor for equal-payment series = 13.5903 at 20 years (from Figure 50-2C)
- \( PWF_{SP} \) = Present-worth factor for single-payment series = 0.4564 at 20 years (from Figure 50-2C)
- \( M_{ac} \) = Annual maintenance cost = $3,000
- \( T \) = Terminal (salvage) value = $20,000

Step 10: Calculate the B/C ratio. Use Equation 50-2.6 to obtain the following:

\[ B/C \text{ Ratio} = \frac{EUAB}{EUAC} = \frac{62,336}{57,529} = 1.0836 \]

Step 11: Calculate the NAB. Use Equation 50-2.7 to obtain the following:

\[ NAB = EUAB - EUAC = 62,336 - 57,529 = 4,807 \]

Comments:

1. The NAB is a positive value as expected because the B/C ratio is greater than 1. This means that, if the proposed improvement were constructed, the projected annual benefits would be $4,807.
2. Because the B/C ratio is greater than 1, this project would be cost effective to construct.

**Example 50-2.2**

Given: S.R. 62, an Urban Collector
Non-freeway 3R Project
Horizontal curve which meets the criteria described in Section 55-4.03, but has a history of accidents as shown in Figure 50-2D, Accident Summary (Example 50-2.1).

Problem: Determine if improving the superelevation at the horizontal curve will be cost-effective.

Solution: The following steps from Section 50-2.03(03) apply.

Step 1: Collect accident data. The accident data is provided in Figure 50-2D.

Step 2: Identify the proposed safety improvement. The selected improvement is to improve the superelevation on the horizontal curve.

Step 3: Determine the service life of improvement. From Figure 50-2B, Service Life, the expected service life for horizontal-alignment change is 20 years.

Step 4: Estimate initial construction and annual maintenance costs. From similar projects, the construction cost is estimated to be $750,000 with annual maintenance after realignment to be $3,000. After 20 years, the terminal (salvage) value is expected to be $20,000.

Step 5: Estimate the assumed accident reduction for each accident type and for each year of service life. The following will apply.

a. From Figure 50-2G, Missouri Accident Reduction Factors, the ARF is 50%. However, because the selected improvement would still have restricted horizontal geometry, an ARF of 30% is assumed for these computations.

b. The APF is assumed to be 2% per year; see Section 50-2.03(02) Item 4, and Figure 50-2F column 2.
c. From Figure 50-2D, the average annual PDO accidents is 5.66 and average annual F/I accidents is 2.33.

d. Using Equation 50-2.2, and Figure 50-2F columns 3 and 4, Accident Reduction Benefits (Example 50-2.2), show the expected number of PDO and F/I accidents to be reduced.

Step 6: Compute accident reduction benefits. The following will apply; see Figure 50-2F.

a. Column 5. Determine the benefits of the reduced number of PDO accidents by multiplying the value in column 3 by $3,000 (from Figure 50-2A) using Equation 50-2.3.

b. Column 6. Determine the benefits of the reduced number of F/I accidents by multiplying the value in column 4 by $37,000 (from Figure 50-2A) using Equation 50-2.3.

c. Column 7. Determine total benefit of the reduced number of accidents by adding columns 5 and 6.

d. Column 8. Determine the present worth factor from Figure 50-2C, 4% Interest Factors for Annual Compounding Interest.

e. Column 9. Determine the present worth of the benefits from the reduced number of accidents by multiplying column 7 by column 8.

f. Total. Determine the total yearly benefits by summing the values in column 9. The total yearly benefit for this example is $508,175.

Step 7: Estimate the secondary benefits. For this example, there are no secondary benefits.

Step 8: Convert gross benefit from Step 6 to EUAB. The CRF factor from Figure 50-2C for 20 years is 0.0736. Using Equation 50-2.4, the EUAB is as follows:

\[ EUAB = 0.0736 \times \$508,175 = \$37,402 \]

Step 9: Convert gross costs to EUAC. Using Equation 50-2.5, the EUAC is as follows:

\[ EUAC = (0.0736) \times \left[ \$750,000 + \$3,000 (13.5903) - \$20,000 (0.4564) \right] = \$57,529 \]
Where:

\[ \text{CRF} = \text{Capital-recovery factor for the last year of the improvement’s service life} = 0.0736 \text{ at 20 years (from Figure 50-2C)} \]

\[ I_c = \text{Initial cost} = $750,000 \]

\[ \text{PWF}_{\text{EPS}} = \text{Present-worth factor for equal payment series} = 13.5903 \text{ at 20 years (from Figure 50-2C)} \]

\[ \text{PWF}_{\text{SP}} = \text{Present-worth factor for single payment series} = 0.4564 \text{ at 20 years (from Figure 50-2C)} \]

\[ \text{M}_{\text{ac}} = \text{Annual maintenance cost} = $3,000 \]

\[ T = \text{Terminal (salvage) value} = $20,000 \]

Step 10: Calculate the B/C ratio using Equation 50-2.6 as follows:

\[ \frac{\text{EUAB}}{\text{EUAC}} = \frac{37,402}{57,529} = 0.6501 \]

Step 11: Calculate the NAB using Equation 50-2.7 as follows:

\[ \text{NAB} = \text{EUAB} - \text{EUAC} = 37,402 - 57,529 = -$20,127 \]

Comments:

1. The NAB is a negative value as expected because the B/C ratio is less than 1. This means that, if the proposed improvement were constructed, the projected annual cost would be $20,127.

2. Because the B/C ratio is considerably less than one, it will not be economically prudent to construct the proposed pavement.

* * * * * * * * * *
50-2.03(05) Accident Reduction Factors

The Department is presently using the accident reduction factors developed by the State of Missouri. These factors are provided in Figure 50-2G.

50-2.04 Safety Benefits Based on Accident Potential (Run-off-the-Road Accident)

It is unusual for a roadside site to have a sufficiently high-accident experience to estimate safety benefits based on accident history. They usually occur at random locations along the highway roadside. However, run-off-the-road accidents in total represent a high proportion of highway accidents. Therefore, roadside hazard improvements may be warranted even if a particular site has never experienced a hazard.

The AASHTO Roadside Design Guide Appendix A provides a methodology to evaluate the cost-effectiveness of a roadside-safety improvement. This methodology will assess the potential for a given hazard to be struck based on pertinent traffic, highway, and hazard characteristics and will allow for the calculation of the cost effectiveness of the alternative countermeasures. It can be used to evaluate individual sites or to evaluate roadside safety for a highway segment (e.g., 1 to 2 miles in length). There is an inherent realization in this approach that a certain number of hazardous locations where a treatment is deemed to be cost effective will never experience an accident, and a certain number of hazardous locations where a treatment is deemed to be not cost effective will, in fact, experience an accident.

The AASHTO methodology establishes the following possible countermeasures in order of desirability.

1. Remove the roadside hazard.
2. Laterally relocate the hazard to a location where the potential for being struck is acceptable.
3. Reduce the severity of the hazard by making it breakaway or by making it traversable.
4. Shield the hazard with guardrail or crash cushion.
5. Do nothing; i.e., leave the hazard unshielded.

The above procedure permits the determination of which countermeasure is the most cost effective.
Chapter 49 provides the Department’s warrants for guardrail and other safety appurtenances. AASHTO Roadside Design Guide Appendix A in conjunction with the Department input data (e.g., accident costs) should be used to determine the appropriate warrant application. Section 49-10.0 provides a step-by-step guide on how to use ROADSIDE (i.e., the ROADSIDE Computer Software Program for Appendix A).

50-3.0 VALUE ENGINEERING [REV. APR. 2016]

50-3.01 General [Rev. Apr. 2016]

Value engineering is not merely a method of cost cutting but a methodology to review alternatives and to suggest choices that still provide a reasonable product without reducing its quality. Value engineering is a proven effective tool for both product improvement and design enhancement. VE can substantially improve design and cost-effectiveness of projects, facilities, operations, procedures and other areas of the transportation program.

The Department must comply with the US Code of Federal Regulations, 23 CFR Part 627, regarding value engineering for each project that utilizes Federal-aid highway funding. A Value Engineering (VE) analysis should be conducted generally around the Public Hearing, during the environmental phase, for complex new or reconstruction projects or 30% plans (Stage 1) for simple projects. The VE analysis must be completed prior to the completion of final design on each applicable State and Local Public Agency (LPA) project. Failure to comply with the VE requirements by PS&E will preclude the use of federal funds and delay the project letting.

Projects that require a VE analysis include the following:

1. Road projects on the National Highway System (NHS) receiving federal assistance with an estimated total cost of $50 million or more. The total cost includes the sum of all engineering, environmental, right of way, Utility, Railroad, and construction costs attributable to the project.

2. Bridge projects on the NHS receiving federal assistance with an estimated total cost of $40 million or more. The total cost includes the sum of all engineering, environmental, right of way, Utility, Railroad, and construction costs attributable to the project.

3. Any major projects, located on or off of the NHS that utilizes Federal-aid highway funding in any contract or phase comprising the major project. A major project is defined as a project with an estimated total cost of $500 million or more.
4. Any project where a VE analysis has not been conducted and a change is made to the project’s scope or design between the final design and the construction letting which results in an increase in the project’s total cost exceeding the thresholds identified above for road and bridge projects.

5. A project determined to be appropriate by FHWA that utilizes Federal-aid highway program funding.

50-3.02 Value Engineering Analysis [Rev. Apr. 2016]

The VE analysis may be completed anytime during the planning, environmental, or design phases of a project as long as there is enough project information available to conduct an effective VE analysis. However, the VE analysis should be completed as early as practical in development of a project to maximize the opportunities for savings.

In accordance with 23 CFR 630.205, all approved VE recommendations must be included in the project’s plans, specifications and estimates (PS&E) prior to authorizing the project for construction. VE analyses are not required for non-NHS bridge projects or for projects delivered using the design/build method of procurement.

If after conducting a VE analysis the project is subsequently split into smaller projects, now under the thresholds shown above, in the design phase or the project is programmed to be completed by the letting of multiple construction projects, an additional VE analysis is not required. However, the project manager may not avoid the requirement to conduct a VE analysis on an applicable project by splitting the project into smaller projects, or programming multiple design or construction projects.

50-3.02(01) Value Engineering Team [Rev. Apr. 2016]

It is the responsibility of the project manager assigned to deliver the project to assemble a multi-disciplinary team to complete the VE analysis and provide recommendations. The team may not include individuals who were directly involved in the planning and development phases of the project, e.g. project manager or designer. The team should be comprised of 3-5 people and led by a different project manager within the Capital Program Management Division. The VE project manager will be responsible for completing the VE analysis in accordance with the Value Engineering Workbook.
50-3.02(02) Value Engineering Workbook [Rev. Apr. 2016]

The Value Engineering Workbook should be completed as soon as possible after Stage 1 review is complete. The project manager will provide the recommendations to the designer for review. The designer should provide comments to the project manager within 15 days. Value Engineering Workbook instructions are available from the Department’s Project Management website.

50-3.02(03) Value Engineering Recommendation and Implementation [Rev. Apr. 2016]

The Value Engineering Recommendation Memo is completed by the VE project manager and should be presented to the Capital Program Management Deputy Commissioner within 15 days of receiving and reconciling the comments from the designer.

The final direction to implement the recommendations will be given by the Deputy Commissioner of Capital Program Management. The Deputy Commissioner will sign off on each recommendation from the memo and include a justification.

The project manager should include the signed VE Recommendation Memo as part of the VE workbook and upload it into ERMS. The following naming convention should be used: FT ValEngStudy DesNumber for Contract Services.

A hard copy of the workbook and recommendations should be distributed to the Project Support Division Director. The changes recommended for implementation are compiled by project and reported to FHWA as part of the annual Value Engineering report.

50-3.03 References [Rev. Apr. 2016]

The following references are available for more detailed information on value engineering techniques and procedures.

1. FHWA Value Engineering Policy, FHWA Order 1311.1B, August 28, 2013.
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<tr>
<th>Route Type</th>
<th>Fatal / Injury *</th>
<th>Property Damage Only</th>
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</thead>
<tbody>
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<td>Interstate Route, Rural</td>
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<td>6,500</td>
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<tr>
<td>Interstate Route, Urban</td>
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<td>6,500</td>
</tr>
<tr>
<td>U.S. or State Route, Rural</td>
<td>78,000</td>
<td>6,500</td>
</tr>
<tr>
<td>U.S. or State Route, Urban</td>
<td>48,000</td>
<td>6,500</td>
</tr>
<tr>
<td>Other Route, Rural</td>
<td>56,500</td>
<td>6,500</td>
</tr>
<tr>
<td>Other Route, Urban</td>
<td>42,500</td>
<td>6,500</td>
</tr>
</tbody>
</table>

* This cost includes property-damage cost.

**ACCIDENT COST PER ACCIDENT**

**In 2001 Dollars**

**Figure 50-2A**
<table>
<thead>
<tr>
<th>Code</th>
<th>Project Description</th>
<th>Service Life</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Intersection Improvement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Channelization, left-turn bay</td>
<td>10</td>
</tr>
<tr>
<td>11</td>
<td>Traffic Signalization</td>
<td>10</td>
</tr>
<tr>
<td>12</td>
<td>Combination of 10 and 11</td>
<td>10</td>
</tr>
<tr>
<td>13</td>
<td>Sight distance improvement</td>
<td>10</td>
</tr>
<tr>
<td>19</td>
<td>Other intersection improvement except structures</td>
<td>10</td>
</tr>
<tr>
<td>1A</td>
<td>Combination of 10 and 19</td>
<td>10</td>
</tr>
<tr>
<td>1B</td>
<td>Combination of 11, 13, 19 and/or 65</td>
<td>10</td>
</tr>
<tr>
<td><strong>Cross Section</strong></td>
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<td></td>
</tr>
<tr>
<td>20</td>
<td>Pavement widening, no lanes added</td>
<td>20</td>
</tr>
<tr>
<td>21</td>
<td>Lanes added without new median</td>
<td>20</td>
</tr>
<tr>
<td>22</td>
<td>Highway divided, new median added</td>
<td>20</td>
</tr>
<tr>
<td>23</td>
<td>Shoulder widening or improvement</td>
<td>20</td>
</tr>
<tr>
<td>24</td>
<td>Combination of 20 and 23</td>
<td>20</td>
</tr>
<tr>
<td>25</td>
<td>Skid treatment, grooving</td>
<td>10</td>
</tr>
<tr>
<td>26</td>
<td>Skid treatment, resurfacing</td>
<td>10</td>
</tr>
<tr>
<td>27</td>
<td>Flattening or clearing side slopes</td>
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<td>29</td>
<td>Other cross section or combination of 20-27</td>
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</tr>
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<td>2A</td>
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<tr>
<td><strong>Structure</strong></td>
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<td></td>
</tr>
<tr>
<td>30</td>
<td>Widening bridge or major structure</td>
<td>20</td>
</tr>
<tr>
<td>31</td>
<td>Replacing bridge or major structure</td>
<td>30</td>
</tr>
<tr>
<td>32</td>
<td>New bridge or major structure, except 34 &amp; 51</td>
<td>30</td>
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<tr>
<td>33</td>
<td>Minor structure</td>
<td>20</td>
</tr>
<tr>
<td>34</td>
<td>Pedestrian over- or under-crossing</td>
<td>30</td>
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<tr>
<td>39</td>
<td>Other structure</td>
<td>20</td>
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<tr>
<td><strong>Alignment</strong></td>
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<tr>
<td>40</td>
<td>Horizontal alignment change, except 52</td>
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<tr>
<td>41</td>
<td>Vertical alignment change</td>
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<td>42</td>
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<tr>
<td>49</td>
<td>Other alignment change</td>
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</tr>
<tr>
<td><strong>Railroad Grade Crossing</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>Add flashing lights</td>
<td>10</td>
</tr>
<tr>
<td>51</td>
<td>Eliminate with new or reconstructed grade separation</td>
<td>30</td>
</tr>
<tr>
<td>52</td>
<td>Elimination by relocating highway or railroad</td>
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<td>53</td>
<td>Illumination</td>
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<td>54</td>
<td>Flashing lights replacing active devices</td>
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<tr>
<td>55</td>
<td>Automatic gates replacing signs</td>
<td>10</td>
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<tr>
<td>56</td>
<td>Automatic gates replacing active devices</td>
<td>10</td>
</tr>
<tr>
<td>57</td>
<td>Signing and marking</td>
<td>10</td>
</tr>
<tr>
<td>58</td>
<td>Crossing-surface treatment</td>
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<tr>
<td>59</td>
<td>Other railroad grade crossing</td>
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**SERVICE LIFE (years)**

*Figure 50-2B*
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<th>Code</th>
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<tr>
<td>60</td>
<td>Traffic signs</td>
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<tr>
<td>61</td>
<td>Breakaway signs or luminaire supports</td>
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</tr>
<tr>
<td>62</td>
<td>Road-edge guardrail</td>
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<td>63</td>
<td>Median barrier</td>
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<td>64</td>
<td>Markings or delineators</td>
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</tr>
<tr>
<td>65</td>
<td>Lighting</td>
<td>15</td>
</tr>
<tr>
<td>66</td>
<td>Improve drainage structures</td>
<td>20</td>
</tr>
<tr>
<td>67</td>
<td>Fencing</td>
<td>10</td>
</tr>
<tr>
<td>68</td>
<td>Impact attenuators</td>
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</tr>
<tr>
<td>69</td>
<td>Other roadside appurtenances</td>
<td>10</td>
</tr>
<tr>
<td>6A</td>
<td>Combination of 60-64</td>
<td>10</td>
</tr>
<tr>
<td>6B</td>
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<td>6E</td>
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<td>6F</td>
<td>Combination of 62, 66, and 69</td>
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<td>6G</td>
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<td>Safety provisions for roadside features and appurtenances</td>
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<td>99</td>
<td>Project not otherwise classified</td>
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<td>Combination of 11 and 64</td>
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<td>9F</td>
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**SERVICE LIFE (years)**

**Figure 50-2B (continued)**
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<thead>
<tr>
<th>Year</th>
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<th>Equal-Payments Series</th>
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<td>Compound Amount</td>
<td>Sinking Fund</td>
<td>Present Worth</td>
<td>Capital Recovery</td>
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<td>6</td>
<td>1.2653</td>
<td>0.7903</td>
<td>6.6330</td>
<td>0.1508</td>
<td>5.2421</td>
<td>0.1908</td>
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<td>7</td>
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4% INTEREST FACTORS FOR ANNUAL COMPOUNDING INTEREST

Figure 50-2C
## Accident Types

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<th>Accident Summary</th>
<th>Year</th>
<th>PD</th>
<th>F/I</th>
<th>Accident Types</th>
<th>H.O.</th>
<th>R.E.</th>
<th>R.A.</th>
<th>S.S.</th>
<th>T.M.</th>
<th>Ped.</th>
<th>L.C.</th>
<th>Other</th>
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Sum of Average PD per Year = 5.6  
Sum of Average F/I per Year = 2.3

Where:

- PD = Property Damage Only
- F/I = Fatal/Injury
- H.O. = Head On
- R.E. = Rear End
- R.A. = Right Angle
- S.S. = Sideswipe
- T.M. = Turning Movement
- Ped. = Pedestrian
- L.C. = Lost Control

### ACCIDENT SUMMARY

(Example 50-2.1)

Figure 50-2D
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<th>Service Year (1)</th>
<th>APF (2)</th>
<th>PDO (3)</th>
<th>F/I (4)</th>
<th>PDO x $3,000 (5)</th>
<th>F/I x $37,000 (6)</th>
<th>Total Benefit (7)</th>
<th>PWF (8)</th>
<th>Adjusted Benefits (9)</th>
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Sum of Average/Yr: PDO = 5.66; F/I = 2.33; APF = 1.02
Summation of Adjusted Total Yearly Benefits = $846,958

**ACCIDENT REDUCTION BENEFITS**
(Example 50-2.1)

Figure 50-2E
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Sum of Average/Yr: PDO = 5.66; F/I = 2.33; APF = 1.02
Summation of Adjusted Total Yearly Benefits = $508,175

ACCIDENT REDUCTION BENEFITS
(Example 50-2.2)

Figure 50-2F
## Accident Reduction Factors (Percent)

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<th>All</th>
<th>Fatal Injury</th>
<th>PDO</th>
<th>Head On</th>
<th>Rear End</th>
<th>Right Angle</th>
<th>Side Swipe</th>
<th>Left Turn</th>
<th>Right Turn</th>
<th>Fixed Object</th>
<th>Pedestrian</th>
<th>Night</th>
<th>Ran Off Road</th>
<th>Wet Pavement</th>
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</thead>
<tbody>
<tr>
<td>Add Left-Turn Lane Without Signals</td>
<td>≥19; 6&lt;sup&gt;a&lt;/sup&gt;</td>
<td>≥80; 54&lt;sup&gt;a&lt;/sup&gt;</td>
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<td>Turn Bay New Left Channelization at Signalized Intersection w/ or w/o Left-Turn Phase</td>
<td>w/o 15; w/ 36&lt;sup&gt;a&lt;/sup&gt;</td>
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<sup>a</sup> On two or more lanes;  
<sup>b</sup> Two lanes;  
<sup>c</sup> Minor street must be 35% or more of total intersection volumes; total intersection volume must be < 8,000 AADT

### MISSOURI ACCIDENT REDUCTION FACTORS

**Figure 50-2G (Continued)**
<table>
<thead>
<tr>
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<sup>a</sup> On two or more lanes;  
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<sup>c</sup> Minor street must be 35% or more of total intersection volumes; total intersection volume must be < 8,000 AADT

MISSOURI ACCIDENT REDUCTION FACTORS

Figure 50-2G
<table>
<thead>
<tr>
<th>Improvement</th>
<th>All</th>
<th>Fatal Injury</th>
<th>PDO</th>
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<th>Rear End</th>
<th>Right Angle</th>
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<th>Right Turn</th>
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<th>Pedestrian</th>
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<td>29&lt;sup&gt;a&lt;/sup&gt;; 41&lt;sup&gt;a&lt;/sup&gt;</td>
<td>≥59&lt;sup&gt;a&lt;/sup&gt;; ≥47&lt;sup&gt;a&lt;/sup&gt; ≥26&lt;sup&gt;a&lt;/sup&gt;</td>
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<sup>a</sup>On two or more lanes; <sup>b</sup>Two lanes
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MISSOURI ACCIDENT REDUCTION FACTORS

Figure 50-2G (Continued)
<table>
<thead>
<tr>
<th>Improvement</th>
<th>Accident Reduction Factors (Percent)</th>
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$^a$ On two or more lanes;  
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**MISSOURI ACCIDENT REDUCTION FACTORS**

Figur**e 50-2G (Continued)
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### Accident Reduction Factors (Percent)

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a On two or more lanes;  
b Two lanes  
c Minor street must be 35% or more of total intersection volumes; total intersection volume must be < 8,000 AADT

### MISSOURI ACCIDENT REDUCTION FACTORS

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<th>Improvement</th>
<th>Accident Reduction Factors (Percent)</th>
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<td>Night</td>
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<td>Run Off Road</td>
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MISSOURI ACCIDENT REDUCTION FACTORS

Figure 50-2G (Continued)
### Accident Reduction Factors (Percent)

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- On two or more lanes;  
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**MISSOURI ACCIDENT REDUCTION FACTORS**

*Figure 50-2G* (Continued)
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<th>Fatal Injury</th>
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<td>v  On two or more lanes;</td>
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<tr>
<td>e  Minor street must be 35% or more of total intersection volumes; total intersection volume must be &lt; 8,000 AADT</td>
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**MISSOURI ACCIDENT REDUCTION FACTORS**

*Figure 50-2G (Continued)*
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<th>Fatal Injury</th>
<th>PDO</th>
<th>Head On</th>
<th>Rear End</th>
<th>Right Angle</th>
<th>Side Swipe</th>
<th>Left Turn</th>
<th>Right Turn</th>
<th>Fixed Object</th>
<th>Pedestrian</th>
<th>Night</th>
<th>Ran Off Road</th>
<th>Wet Pavement</th>
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</table>

$^a$ On two or more lanes

$^b$ Two lanes

$^c$ Minor street must be 35% or more of total intersection volumes; total intersection volume must be < 8,000 AADT

**MISSOURI ACCIDENT REDUCTION FACTORS**

**Figure 50-2G** (Continued)
CHAPTER 53

Geometric Design Tables (New Construction/Reconstruction)

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

<table>
<thead>
<tr>
<th>Design Memorandum</th>
<th>Revision Date</th>
<th>Sections Affected</th>
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<tr>
<td>14-10</td>
<td>Jul. 2014</td>
<td>53-1.0, Figures 53-1 through 53-9</td>
</tr>
</tbody>
</table>
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Figure  Title

CHAPTER 53

GEOMETRIC DESIGN TABLES
(NEW CONSTRUCTION/RECONSTRUCTION)

This chapter provides the Department’s criteria for the design of a new construction or reconstruction (4R) project. The values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein except as follows.

1. The *Green Book* minimum values may not be used to supersede State or Federal code requirements, e.g. National Truck Network, American with Disabilities Act (ADA).
   a. Highways that are on the National Truck Network must use 12-ft lanes. In Indiana, the National Truck Network is comprised of those routes designated as Federal-Aid primary as of June 1, 1991. The National Truck Network is available as a separate layer on the INDOT Roadway Inventory map at http://gis.in.gov/apps/DOT/RoadwayInventory/.

2. Vertical clearance requirements for new and replaced bridges, sign trusses, and pedestrian structures must include an additional 6” for consideration of future resurfacing.

3. Ramp design requirements, including acceleration and deceleration length are as described in Chapter 48. Ramp reconstruction requirements as part of a 3R or Partial 4R Freeway project are as described in Chapter 54.

4. The superelevation rate should not exceed \( e_{\text{max}} = 8\% \) due to the prevalence of snow and ice.
53-1.0 GEOMETRIC DESIGN TABLE FIGURES [REV. JUL 2014]

The following should be considered in the use the figures.

1. **Project Scope of Work (Freeway).** The geometric design criteria shown in Figure 53-1 apply to new construction or complete reconstruction of a freeway. The Department has adopted separate criteria for a 3R project or a partial 4R project on a freeway. See Chapter 54. Chapters 40 and 54 provide definitions for the freeway-project scope of work, which will determine which set of criteria should be used for project design.

2. **Project Scope of Work (Non-Freeway).** The geometric design criteria shown in Figures 53-2 through 53-9 apply to a new construction or reconstruction (4R) project on a non-freeway. The Department has adopted separate criteria for the geometric design of a 3R non-freeway project. See Chapter 55. Chapter 40 provides definitions for the non-freeway-project scope of work, which will determine which set of criteria should be used for project design.

3. **Functional Classification.** The selection of design values depends on the functional classification of the highway facility. This is discussed in Section 40-1.01. Functional-classification maps for all public roads are available from the Planning Division. See Section 40-1.01 for definitions of the functional classifications.

4. **Urban Design Subcategories.** Within an urbanized or urban area, the selection of design values depends on the design subcategory of the facility. Separate criteria are provided for suburban, intermediate, and built-up subcategories. These classifications are defined as follows.

   a. **Suburban.** This type of area is located at the fringe of an urbanized or small urban area. The predominant character of the surrounding environment is residential, but it may include a considerable number of commercial establishments, especially strip development along a suburban arterial. There may also be a few industrial parks. On a suburban road or street, a motorist has a significant degree of freedom, but nonetheless he or she must also devote some of their attention to entering and exiting vehicles. Roadside development is characterized by low to moderate density. Pedestrian activity may or may not be a significant design factor. Right of way is often available for roadway improvements.

   A local or collector street is located in a residential area, but may also serve a commercial area. The posted speed limit ranges between 30 and 50 mph. The
majority of intersections will have stop or yield control, but there will be an occasional traffic signal. A suburban arterial will have strip commercial development and perhaps a few residential properties. The posted speed limit ranges between 35 and 55 mph, and there will usually be a few signalized intersections along the arterial.

b. Intermediate. As the name implies, an intermediate area is between a suburban and a built-up area. The surrounding environment may be either residential, commercial, or industrial or a combination of these. The extent of roadside development will have a significant impact on the selected speeds of motorists. The increasing frequency of intersections is also a control on average speed. Pedestrian activity has now become a significant design consideration, and sidewalks and cross walks at intersections are common. The available right of way will restrict the practical extent of roadway improvements.

A local or collector street has a posted speed limit ranging between 30 and 45 mph. The frequency of signalized intersections has increased substantially if compared to a suburban area. An arterial will have intensive commercial development along its roadside. The posted speed limit ranges between 35 and 50 mph. Such an arterial has several signalized intersections per mile.

c. Built-up. This type of area refers to the central business district within an urbanized or small urban area. The roadside development has a high density and is often commercial. However, a substantial number of roads and streets pass through a high-density environment (e.g. apartment complexes, row houses). Access to property is the primary function of the road network. Pedestrian considerations may be as important as vehicular considerations, especially at intersections. Right of way for roadway improvements is usually not available.

Because of the high density of development, the distinction between the functional classifications (local, collector, or arterial) becomes less important when considering signalization and speeds. The primary distinction among the three functional classes is often the relative traffic volume and, therefore, the number of lanes. As many as half the intersections may be signalized. The posted speed limit ranges between 25 and 35 mph.

If the area is rural in character (e.g., a sparsely-populated area without a gridlike street system), it may be appropriate to use the rural-area design criteria though the facility is urban.
5. **Rural-Area Figures.** These do not provide design criteria for sub-categories. However, there are many rural facilities which pass through relatively built-up, but unincorporated, areas. It may be inappropriate to use the rural-area design criteria. The designer may, as an option, use the suburban criteria for a functional classification (e.g., arterial) in a relatively built-up rural area. Therefore, if the area is urban in character (e.g., a densely populated area with a grid-like street system) it may be appropriate to use the urban-area design criteria even though the facility is rural. This decision will be documented in the Engineer’s Report (see Chapter 7).

6. **Cross-Section Elements.** Some of the cross-section elements included in a figure (e.g., sidewalk width) are not automatically warranted in the project design. The values will only apply after the decision has been made to include the element in the highway cross section.

7. **Manual Section References.** The figures are intended to provide a concise listing of design values for easy use. However, the designer should review the *Manual* section references for greater insight into the design elements.

8. **Footnotes.** The figures include many footnotes, which are identified by a number in parentheses, e.g., (6). The information in the footnotes is critical to the proper use of the figures.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Rural</th>
<th>Urban</th>
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<td><strong>Design Controls</strong></td>
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<td>20 Years</td>
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<td>40-5.0</td>
<td>Full Control</td>
<td>Full Control</td>
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<td>Level of Service</td>
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<td>Desirable: B Minimum: C (2)</td>
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<td>Paved Width ≤ 4 ft: 2% Paved Width &gt; 4 ft: 4%</td>
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<td>6:1 to Clear Zone; 3:1 max. to Toe</td>
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<td>New: 17.5 ft Existing: 17 ft</td>
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D: Desirable  M: Minimum.
* Level One controlling criterion, see page 2 of 4

GEOMETRIC DESIGN CRITERIA FOR FREEWAY, 4R PROJECT
Figure 53-1 (Page 1 of 4)
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<td></td>
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<td></td>
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<td></td>
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<td></td>
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<td></td>
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<td>Minimum: 0.0%</td>
</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.
(1) **Design Speed.** A 50 mph design speed may be considered in a restricted urban area.

(2) **Level of Service.** A minimum Level of Service of D may be used on an urban reconstruction project.

(3) **Surface Type.** The pavement-type selection will be determined by the Pavement Engineering Division.

(4) **Shoulder Width, Right.** The following will apply.
   a. The shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. Where the number of trucks exceeds 250 DDHV, a 12-ft paved width should be used.
   c. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(5) **Shoulder Width, Left.** The following will apply.
   a. The usable shoulder width is equal to the paved shoulder width. The desirable guardrail offset is 2 ft from the usable-shoulder width. See Section 49-4.0 for more information.
   b. Where there are 3 or more lanes in one direction and the volume of trucks exceed 250 DDHV, a 12 ft width should be used.
   c. For a left shoulder of 4 ft or wider, the usable shoulder width will be 1 ft more than the paved-shoulder width.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(6A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

(7) **Auxiliary-Lane Shoulder Width, Right.** On a reconstruction project, a 6-ft width may be used.

(8) **Clear-Zone Width.** This will vary according to design speed, traffic volume, side slopes, and horizontal curvature. See Section 49-2.0.

(9) **Side Slopes.** Value is for new construction. See Sections 45-3.0 for more information. For a reconstruction project, see Section 49-3.0.

(10) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(11) **Ditch Width.** A V-ditch should be used in a rock cut.

(12) **Backslope.** For an earth cut of 10 ft or deeper, the first horizontal 20 ft of the backslope will be sloped at a rate of 4:1. Then, a slope rate of 3:1 is normally used to the natural ground line. The backslope for a rock cut will vary according to the height of cut and the geotechnical requirements. See Sections 45-3.0 and 107-6.01.
(13) **Structural Capacity, New or Reconstructed Bridge.**
   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck-loading configuration.

(14) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge railing offset (see Figure 402-6H).

(15) **Vertical Clearance, Freeway Under.** The following will apply.
   a. Table value includes an additional 6 in. allowance for future overlays.
   b. A 14-ft clearance may be used in an urban area where an alternate freeway facility with a 16-ft clearance is available.
   c. Vertical clearance applies from usable edge to usable edge of shoulders.

(16) **Vertical Clearance, Freeway Over Railroad.** See Section 402-6.01(03) for additional information on railroad clearance under a highway.

(17) **Decision Sight Distance.** Value is for the avoidance maneuver (speed/path/direction change). See Section 42-2.0.

(18) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius.

(19) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance. Sometimes, the stopping-sight-distance value for a truck should be considered. See the discussion in Section 43-4.0.

(20) **Maximum Grade.** A grade of 1% steeper may be used in a restricted urban area where development precludes the use of a flatter grade. A downgrade of 1% steeper may also be used for a one-way roadway.

(21) For a bridge of 200 ft or longer that is to remain in place, the minimum width of each shoulder is 4 ft. This requirement does not apply to a bridge-deck replacement.
### GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIAL

**New Construction or Reconstruction**

Figure 53-2 (Page 1 of 4)

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2 Lanes</th>
<th>4 or More Lanes</th>
</tr>
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<tbody>
<tr>
<td><strong>Design Controls</strong></td>
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<tr>
<td>Design-Year Traffic, AADT</td>
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<td><strong>Undivided</strong></td>
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<td>400 ≤ AADT</td>
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<td>≥ 2000</td>
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<td>Design Forecast Period</td>
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<td>Level: 60 – 70; Rolling: 50 – 60</td>
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<td>Partial Control / None</td>
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<td>Level of Service</td>
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<thead>
<tr>
<th>Section</th>
<th>Travel Lane</th>
<th>Shoulder (3)</th>
<th>Cross Slope</th>
<th>Auxiliary Lane</th>
<th>Shoulder Width (6)</th>
<th>Median Width</th>
<th>Clear-Zone Width</th>
<th>Side Slopes (9)</th>
<th>Median Slopes</th>
<th>Bridges</th>
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<td>11 ft (3b)</td>
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<td></td>
<td>*Width Paved</td>
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<td>6 ft</td>
<td>10 ft (3b)</td>
<td>10 ft (3b)</td>
<td>Right: 10 ft (3b)</td>
<td>Left: 4 ft (3e)</td>
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<td>Paved Width ≤ 4 ft: 2%; Paved Width &gt; 4 ft: 4%</td>
<td>Paved Width ≤ 4 ft: 2%; Paved Width &gt; 4 ft: 4%</td>
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<td>Desirable: 12 ft; Minimum: 11 ft</td>
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<td>New or Reconstructed Bridge</td>
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<td>HL-93 (13)</td>
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<td>Travelway Plus 2 ft on Each Side</td>
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<td>23 ft</td>
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</tbody>
</table>

* Level One controlling criterion, see page 2 of 4

** An arterial of 4 or more lanes on a new location should be designed as Divided.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>50 mph</th>
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<th>70 mph</th>
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<td>495 ft</td>
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<td>730 ft</td>
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<tr>
<td>Decision Sight Distance</td>
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<td>865 ft</td>
<td>990 ft</td>
<td>1105 ft</td>
</tr>
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<td>535 ft</td>
<td>610 ft</td>
<td>780 ft</td>
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<td>Passing Sight Distance</td>
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<td>1985 ft</td>
<td>2135 ft</td>
<td>2480 ft</td>
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<td>Decision Sight Distance</td>
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<td>865 ft</td>
<td>990 ft</td>
<td>1105 ft</td>
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<td>Intersection Sight Distance, -3% to +3% (20)</td>
<td>46-10.0</td>
<td>P: 630 ft; SUT: 780 ft</td>
<td>P: 730 ft; SUT: 890 ft</td>
<td>P: 840 ft; SUT: 1020 ft</td>
<td>P: 1030 ft; SUT: 1240 ft</td>
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<td>*Minimum Radius, e=8%</td>
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<td>1000 ft</td>
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<td>*Vertical Curvature, K-value</td>
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<td>114</td>
<td>151</td>
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<td>*Maximum Grade (19)</td>
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<td>3%</td>
<td>3%</td>
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<td>5%</td>
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* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s A Policy on Geometric Design of Highways and Streets (the Green Book) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply to a route either on or off the National Highway System, regardless of funding source.
(1) **Design Speed.** The minimum design speed should equal the minimum value from the table or the anticipated posted speed limit after construction, whichever is greater. The legal speed limit is 60 mph on a non-posted divided highway.

(2) **Surface Type.** The pavement-type selection will be determined by the INDOT Office of Pavement Engineering.

(3) **Shoulder.** The following will apply.
   a. If there are 3 or more lanes in each direction and there is a median barrier, a 10 ft paved shoulder and a 2 ft offset is required.
   b. For new construction with \(2000 \leq \text{AADT} < 5000\), this may be 8 ft. On a reconstruction project, the usable shoulder width may be 10 ft, and the paved shoulder width may be 8 ft.
   c. The shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   d. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.
   e. If there are 3 or more lanes in each direction, a full-width shoulder, 11 ft usable and 10 ft paved, is desirable.
   f. If curbs are to be used, the criteria described in Figure 53-6 or 53-7 should be applied.

(4) **Cross Slope, Travel Lanes.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place. Where three or more lanes are sloped in the same direction, each successive pair of lanes may have an increased sideslope.

(4A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more specific information.

(5) **Auxiliary Lane, Lane Width.** Truck climbing-lane width is 12 ft.

(6) **Auxiliary Lane, Shoulder Width.** At a minimum, a 2 ft shoulder may be used adjacent to an auxiliary lane. At a minimum, the shoulder adjacent to a truck climbing lane is 4 ft.

(7) **Median Width, Flush.** Value is for new construction. A median of 25 ft or narrower should be avoided at an intersection. A median wider than 60 ft is undesirable at a signalized intersection or at an intersection that may become signalized in the foreseeable future. On a reconstruction project, the minimum flush-median width is 14 ft for a roadway with left-turn lanes, or 22 ft for a roadway with concrete median barrier.

(8) **Clear-Zone Width.** This will vary according to design speed, traffic volume, side slopes, and horizontal curvature. See Section 49-2.0.

(9) **Side Slope.** Value is for new construction. See Sections 45-3.0 for more information. For a reconstruction project, see Section 49-3.0.

(10) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(11) **Ditch Width.** A V-ditch should be used in a rock cut.

**GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIAL**
*(New Construction or Reconstruction)*
Figure 53-2 (Page 3 of 4)
(12) **Backslope.** The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Sections 45-3.0 and 107-6.01.

(13) **Structural Capacity, New or Reconstructed Bridge.** The following will apply.
   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck-loading configuration.

(14) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(15) **Vertical Clearance, Arterial Under.** Value includes an additional 6 in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulders.

(16) **Vertical Clearance, Arterial Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(17) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius.

(18) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. Sometimes, the stopping-sight-distance value for a truck will apply. See the discussion in Section 43-4.0.

(19) **Maximum Grade.** A grade of 1% steeper may be used for a downgrade on a one-way roadway.

(20) **Intersection Sight Distance.** For a left turn onto a 2-lane road: P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2 Lanes</th>
</tr>
</thead>
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<tr>
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<td>&lt; 400</td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>40-2.02</td>
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</tr>
<tr>
<td>*Design Speed, mph (2)</td>
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<td>35 - 55</td>
</tr>
<tr>
<td>Access Control</td>
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</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td></td>
</tr>
</tbody>
</table>

### Design Controls

| Travel Lane Width | 45-1.01 | D: 12 ft; M: 10 ft | D: 12 ft; M: 11 ft | D: 12 ft; M: 11 ft (20) | 12 ft |
| Shoulder Width Usable | 45-1.02 | 4 ft | 6 ft | 8 ft | 10 ft |
| Shoulder Width Paved | 45-1.02 | 2 ft | 4 ft | 6 ft | 8 ft |
| Typical Surface Type (3) | Chp. 304 | Asphalt / Concrete |

### Shoulder (4)

| Typical Surface Type (3) | Chp. 304 |
| Shoulder Width (5A) | 45-1.02 |
| Paved Width ≤ 4 ft: 2%; Paved Width > 4 ft: 4% |

### Cross Slope

| *Travel Lane (5) | 45-1.01 | 2% |
| Shoulder (5A) | 45-1.02 | Desirable: 12 ft; Minimum: 11 ft |

### Auxilary Lane

| Lane Width | 45-1.03 | Des: Same as Through Lanes; Min: 11 ft |
| Shoulder Width (6) | Same as Next to Travel Lane |

### Clear-Zone Width

| 49-2.0 |

### Side Slopes (8)

| Foreslope | 45-3.0 | Des: 6:1; Max: 4:1 (9) |
| Ditch Width | 4 ft (10) |
| Backslope | 4:1 for 20 ft; 3:1 Max. to Top (11) |

### New or Reconstructed Bridge

| *Structural Capacity | Chp. 403 | HL-93 (12) |
| *Clear-Roadway Width (13) | 45-4.01 | Full Paved Approach Width |

### Existing Bridge to Remain in Place

| *Structural Capacity | Chp. 72 | HS-15 |
| *Clear-Roadway Width (14) | 45-4.01 | 22 ft | 22 ft | 24 ft | 28 ft |

### Bridges

| New or Replaced Overpassing Bridge (15) | 44-4.0 | 14.5 ft |
| Existing Overpassing Bridge | 14 ft |

### Vertical Clearance, Collector Over Railroad (16)

| Chp. 402-6.01 | 23 ft |

---

D or Des: Desirable; M or Min: Minimum

* Level One controlling criterion, see page 2 of 4

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GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, STATE ROUTE

(New Construction or Reconstruction)

Figure 53-3 (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2 Lanes</th>
<th>40 mph</th>
<th>45 mph</th>
<th>50 mph</th>
<th>55 mph</th>
<th>60 mph</th>
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<tr>
<td>Alignment Elements</td>
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</tr>
<tr>
<td>*Stopping Sight Distance</td>
<td>42-1.0</td>
<td>305 ft</td>
<td>360 ft</td>
<td>425 ft</td>
<td>495 ft</td>
<td>570 ft</td>
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<tr>
<td>Decision Sight Distance</td>
<td>42-2.0</td>
<td>600 ft</td>
<td>675 ft</td>
<td>750 ft</td>
<td>865 ft</td>
<td>990 ft</td>
<td></td>
</tr>
<tr>
<td>Speed / path / direction change</td>
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<tr>
<td>Decision Sight Distance</td>
<td>42-2.0</td>
<td>600 ft</td>
<td>675 ft</td>
<td>750 ft</td>
<td>865 ft</td>
<td>990 ft</td>
<td></td>
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<tr>
<td>Stop Maneuver</td>
<td>42-3.0</td>
<td>1470 ft</td>
<td>1625 ft</td>
<td>1835 ft</td>
<td>1985 ft</td>
<td>2135 ft</td>
<td></td>
</tr>
<tr>
<td>Passing Sight Distance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (21)</td>
<td>46-10.0</td>
<td>P: 440 ft</td>
<td>P: 500 ft</td>
<td>P: 630 ft</td>
<td>P: 730 ft</td>
<td>P: 840 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SUT: 560 ft</td>
<td>SUT: 630 ft</td>
<td>SUT: 780 ft</td>
<td>SUT: 890 ft</td>
<td>SUT: 1020 ft</td>
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</tr>
<tr>
<td>*Minimum Radius, e=8%</td>
<td>43-2.0</td>
<td>410 ft</td>
<td>590 ft</td>
<td>750 ft</td>
<td>1000 ft</td>
<td>1290 ft</td>
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<tr>
<td>*Superelevation Rate</td>
<td>43-3.0</td>
<td>e_{max} = 8% (17)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>*Horizontal Sight Distance</td>
<td>43-4.0</td>
<td>(18)</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>*Vertical Curvature, K-value</td>
<td></td>
<td>Crest</td>
<td></td>
<td></td>
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<td>Sag</td>
<td>64</td>
<td>79</td>
<td>96</td>
<td>115</td>
<td>136</td>
</tr>
<tr>
<td>*Maximum Grade (19)</td>
<td>44-1.02</td>
<td>7%</td>
<td>7%</td>
<td>6%</td>
<td>6%</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>Desirable: 0.5%</td>
<td>Minimum: 0.0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s A Policy on Geometric Design of Highways and Streets (the Green Book) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply to each project regardless of funding source.
(1) **Design Forecast Year.** If the DHV is less than 100 (based on a 20-year projection) the current AADT may be used for design.

(2) **Design Speed.** The minimum design speed should equal the minimum value from the table or the anticipated posted speed limit after construction, whichever is higher. The legal speed limit is 55 mph on a non-posted highway.

(3) **Surface Type.** The pavement-type selection will be determined by the INDOT Office of Pavement Engineering.

(4) **Shoulder Width.** The following will apply.
   a. The shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.
   c. If curbs are to be used, the criteria described in Figure 53-8 should be applied.

(5) **Cross Slope, Travel Lanes.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(5A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

(6) **Auxiliary Lane, Shoulder Width.** At a minimum, a 2 ft width may be used adjacent to an auxiliary lane.

(7) **Clear-Zone Width.** This will vary according to design speed, traffic volume, side slopes, and horizontal curvature. See Section 49-2.0.

(8) **Side Slope.** Value is for new construction. See Sections 45-3.0 for more information. For a reconstruction project, see Section 49-3.0.

(9) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(10) **Ditch Width.** A V-ditch should be used in a rock cut. See Sections 45-3.0 and 107-6.01.

(11) **Backslope.** The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Section 107-6.01 for typical rock-cut sections.
(12) **Structural Capacity, New or Reconstructed Bridge.** The following will apply.
   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck-loading configuration.

(13) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable-shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(14) **Width, Existing Bridge to Remain in Place.** Clear-roadway width will be at least equal to the approach traveled-way width or the table value, whichever is greater.

(15) **Vertical Clearance, Collector Under.** Value includes an additional 6-in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulders.

(16) **Vertical Clearance, Collector Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(17) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius.

(18) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. See Section 43-4.0.

(19) **Maximum Grade.** For a grade along a longitudinal distance of less than 480 ft (PVT to PVC), a one-way downgrade, or a road with AADT < 400, the maximum grade may be up to 2% steeper than the table value.

(20) Use 12 ft if V = 55 mph.

(21) **Intersection Sight Distance.** For a left turn onto a 2-lane roadway. P = Passenger car; SUT = single unit truck. See Figure 46-10G for values for a combination truck.
### GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, LOCAL-AGENCY ROUTE

(New Construction or Reconstruction)

Figure 53-4 (Page 1 of 4)
### Alignment Elements

#### Design Speed
- Design Speed
  - 30 mph
  - 35 mph
  - 45 mph
  - 50 mph
  - 55 mph
  - 60 mph

#### *Stopping Sight Distance*
- 42-1.0
  - 200 ft
  - 250 ft
  - 360 ft
  - 425 ft
  - 495 ft
  - 570 ft

#### Decision Sight Distance
- 42-2.0
  - 450 ft
  - 525 ft
  - 675 ft
  - 750 ft
  - 865 ft
  - 990 ft

#### Stop Maneuver
- 42-2.0
  - 220 ft
  - 275 ft
  - 395 ft
  - 465 ft
  - 535 ft
  - 610 ft

#### Passing Sight Distance
- 42-3.0
  - 1090 ft
  - 1280 ft
  - 1625 ft
  - 1835 ft
  - 1985 ft
  - 2135 ft

#### Intersection Sight Distance, -3% to +3% (19)
- 46-10.0
  - P: 330 ft
  - SUT: 420 ft
  - P: 390 ft
  - SUT: 490 ft
  - P: 500 ft
  - SUT: 630 ft
  - P: 630 ft
  - SUT: 780 ft
  - P: 730 ft
  - SUT: 890 ft
  - P: 840 ft
  - SUT: 1020 ft

#### *Minimum Radius, e=8%*
- 43-2.0
  - 270 ft
  - 410 ft
  - 590 ft
  - 750 ft
  - 1000 ft
  - 1290 ft

#### *Superelevation Rate*
- 43-3.0
  - \( e_{\text{max}} = 8\% \) (16)

#### *Horizontal Sight Distance*
- 43-4.0
  - (17)

#### *Vertical Curvature, K-value*
- 44-3.0
  - Crest
    - 19
    - 29
    - 61
    - 84
    - 114
    - 151
  - Sag
    - 37
    - 49
    - 79
    - 96
    - 115
    - 136

#### *Maximum Grade (18)*
- 44-1.02
  - Level
    - 7%
    - 7%
    - 6%
    - 6%
    - 5.5%
    - 5%
  - Rolling
    - 9%
    - 8%
    - 7%
    - 7%
    - 6.5%
    - 6%

#### Minimum Grade
- 44-1.03
  - Desirable: 0.5%; Minimum: 0.0%

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* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply only to a federal-aid project.

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**GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, LOCAL-AGENCY ROUTE**

*(New Construction or Reconstruction)*

**Figure 53-4 (Page 2 of 4)**
3) Design Speed. The minimum design speed should equal the minimum value or the anticipated posted speed limit after construction, whichever is greater. The legal speed limit is 55 mph on a non-posted highway.

4) Travel-Lane Width. The following will apply.
   a. Use an 11-ft width if the design speed is 55 mph.
   b. Use a 12-ft width if the design speed is 55 mph.

5) Shoulder Width. The following will apply.
   a. If guardrail is required, the minimum width is 4 ft.
   b. Usable-shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.
   c. If curbs are to be used, the criteria described in Figure 53-8 should be applied.

6) Cross Slope, Travel Lanes. Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

6A) Cross Slope, Shoulder. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

7) Clear-Zone Width. This will vary according to design speed, traffic volume, side slopes, and horizontal curvature. See Section 49-2.0.

8) Side Slope. Value is for new construction. See Section 45-3.0 for more information. For a reconstruction project, see Section 49-3.0.

9) Foreslope. See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

10) Ditch Width. A V-ditch should be used in a rock cut.

11) Backslope. The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Sections 45-3.02 and 107-6.02 for typical rock-cut sections.
(12) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(13) **Width, Existing Bridge to Remain in Place.** Clear-roadway width will be at least equal to the approach traveled-way width or the table value, whichever is greater. For a bridge longer than 100 ft, the value does not apply. The acceptability of such a bridge will be assessed individually.

(14) **Vertical Clearance, Collector Under.** Value includes an additional 6 in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulders.

(15) **Vertical Clearance, Collector Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(16) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius.

(17) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. See Section 43-4.0.

(18) **Maximum Grade.** For a grade along a longitudinal distance of less than 480 ft (PVT to PVC), a one-way downgrade, or a road with AADT < 400, the maximum grade may be up to 2% steeper than the table value.

(19) **Intersection Sight Distance.** For a left turn onto a 2-lane roadway: P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.

---

**GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, LOCAL-AGENCY ROUTE**

*(New Construction or Reconstruction)*

Figure 53-4 (Page 4 of 4)
<table>
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<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2 Lanes</th>
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<tbody>
<tr>
<td>Design Controls</td>
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<td>Design-Year Traffic, AADT</td>
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<td>40-2.0</td>
<td>Desirable: B, Minimum: D</td>
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<tr>
<td>Cross-Section Elements</td>
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<td></td>
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<tr>
<td>Travel Lane</td>
<td>*Width</td>
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<td>*Width Usable</td>
<td>45-1.02</td>
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<td></td>
<td>Typical Surface Type</td>
<td>Chp. 304</td>
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<td>*Travel Lane (6)</td>
<td>45-1.01</td>
</tr>
<tr>
<td></td>
<td>Shoulder (6A)</td>
<td>45-1.02</td>
</tr>
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<td>Auxiliary Lane</td>
<td>Lane Width</td>
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<tr>
<td></td>
<td>Shoulder Width</td>
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<td>Clear-Zone Width</td>
<td>49-2.0</td>
<td>(7)</td>
</tr>
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<td>Side Slopes</td>
<td>Cut</td>
<td>45-3.0</td>
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<tr>
<td></td>
<td>Ditch Width</td>
<td>Des: 4 ft; Min: 0.0 ft</td>
</tr>
<tr>
<td></td>
<td>Backslope</td>
<td>4:1 (V ≥ 60); 3:1 (V ≤ 50) (9)</td>
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<td>Fill</td>
<td>Desirable: 4:1; Maximum: 3:1</td>
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<td>45-3.0</td>
</tr>
<tr>
<td></td>
<td>&gt;30 ft Height</td>
<td>3:1</td>
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<td>Bridges</td>
<td>New or Reconstructed Bridge</td>
<td>*Structural Capacity</td>
</tr>
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<td>*Clear-Roadway Width (10)</td>
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<td>Existing Bridge to Remain in Place</td>
<td>*Structural Capacity</td>
<td>Chp. 72</td>
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<td>*Clear-Roadway Width (11)</td>
<td>45-4.01</td>
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<tr>
<td>*Vertical Clearance, Local Road Under</td>
<td>New or Replaced Overpassing Bridge (12)</td>
<td>44-4.0</td>
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<td>Existing Overpassing Bridge</td>
<td>14 ft</td>
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<td></td>
<td>Vertical Clearance, Local Road Over Railroad) (13)</td>
<td>Chp. 402-6.01</td>
</tr>
</tbody>
</table>


* Level One controlling criterion, see page 2 of 4.

GEOMETRIC DESIGN CRITERIA FOR RURAL LOCAL ROAD
(New Construction or Reconstruction)
Figure 53-5 (Page 1 of 4)
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<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2 Lanes</th>
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<tr>
<td><strong>Design Speed</strong></td>
<td>42-1.0</td>
<td>20 mph 25 mph 30 mph 35 mph 45 mph 50 mph 55 mph</td>
</tr>
<tr>
<td><strong>Stopping Sight Distance</strong></td>
<td>42-2.0</td>
<td>115 ft 155 ft 200 ft 250 ft 360 ft 425 ft 495 ft</td>
</tr>
<tr>
<td><strong>Decision Sight Distance</strong></td>
<td>42-2.0</td>
<td>300 ft 375 ft 450 ft 525 ft 675 ft 750 ft 865 ft</td>
</tr>
<tr>
<td><strong>Passing Sight Distance</strong></td>
<td>42-3.0</td>
<td>710 ft 900 ft 1090 ft 1280 ft 1625 ft 1835 ft 1985 ft</td>
</tr>
<tr>
<td><strong>Intersection Sight Distance</strong></td>
<td>46-10.0</td>
<td>220 ft 280 ft 330 ft 390 ft 500 ft 550 ft 610 ft</td>
</tr>
<tr>
<td><strong>Minimum Radius, e=8%</strong></td>
<td>43-2.0</td>
<td>80 ft 135 ft 215 ft 315 ft 590 ft 760 ft 960 ft</td>
</tr>
<tr>
<td><strong>Superelevation Rate</strong></td>
<td>43-3.0</td>
<td>$e_{\text{max}}=8%$ (14)</td>
</tr>
<tr>
<td><strong>Horizontal Sight Distance</strong></td>
<td>43-4.0</td>
<td>(15)</td>
</tr>
<tr>
<td><strong>Vertical Curvature, K-value</strong></td>
<td>44-3.0</td>
<td>7 12 19 29 61 84 114</td>
</tr>
<tr>
<td>Crest</td>
<td></td>
<td>17 26 37 49 79 96 115</td>
</tr>
<tr>
<td>Sag</td>
<td></td>
<td>8% 7% 7% 7% 7% 6% 5.5%</td>
</tr>
<tr>
<td><strong>Maximum Grade</strong></td>
<td>44-1.02</td>
<td>11% 11% 10% 9% 9% 8% 7%</td>
</tr>
<tr>
<td>Level</td>
<td></td>
<td>Desirable: 0.5%; Minimum: 0.0%</td>
</tr>
<tr>
<td>Rolling</td>
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<td></td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td></td>
</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply only to a federal-aid project.
(3) **Design Speed.** The minimum design speed should equal the minimum value or the anticipated posted speed limit after construction, whichever is greater. The legal speed limit is 55 mph on a non-posted highway.

(4) **Travel Lane Width.** The following will apply.
   a. Use 11-ft lanes where $V \geq 50$ mph.
   b. Use 12-ft lanes where $V \geq 55$ mph.

(5) **Shoulder Width.** The following will apply.
   a. For $400 \leq \text{AADT} < 1500$, the shoulder width may be 4 ft.
   b. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.
   c. If curbs are to be used, the criteria described in Figure 53-8 should be applied.

(6) **Cross Slope, Travel Lanes.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(6A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

(7) **Clear-Zone Width.** This will vary according to design speed, traffic volume, side slopes, and horizontal curvature. See Section 49-2.0. For a design speed of lower than 50 mph, a 10 ft clear-zone width may be used.

(8) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(9) **Backslope.** The backslopes for a rock cut will vary according to the height of the cut and the geotechnical requirements.
(10) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(11) **Width, Existing Bridge to Remain in Place.** Minimum clear-roadway width of 2 ft narrower than the value may be used on a road with few trucks. The clear-roadway width should be at least the same width as the approach travelway. For a one-lane bridge, the width may be 18 ft. For a bridge longer than 100 ft, the value does not apply. The acceptability of each such bridge will be assessed individually.

(12) **Vertical Clearance, Local Road Under.** Value includes an additional 6 in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulders.

(13) **Vertical Clearance, Local Road Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under highway.

(14) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius.

(15) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. See Section 43-4.0.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Design Value (By Type of Area)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design Controls</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>40-2.02</td>
<td>Suburban: 20 Years</td>
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<tr>
<td>Design Speed, mph (1)</td>
<td>40-3.0</td>
<td>Curbed: 45-55</td>
</tr>
<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>Partial Control / None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Des: B; Min: C</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>45-1.04</td>
<td>Optional (2)</td>
</tr>
<tr>
<td><strong>Cross-Section Elements</strong></td>
<td></td>
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<td>Travel Lane</td>
<td>45-1.01</td>
<td>Curbed: 12 ft</td>
</tr>
<tr>
<td>Typical Surface Type (4)</td>
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<td>Asphalt / Concrete</td>
</tr>
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<td>Shoulder</td>
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Des: Desirable, Min: Minimum.

* Level One controlling criterion, see page 2 of 4
### Design Element

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<td>HL-93</td>
<td>HL-93</td>
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<td>HS-20</td>
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<tr>
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<td>16.5 ft (22b)</td>
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<td>14 ft</td>
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<td>Sign Truss / Pedestrian Bridge (22a)</td>
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<td>17.5 ft; Existing: 17 ft</td>
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### Bridges

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<td>HS-20</td>
<td>HS-20</td>
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<tr>
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<td>14 ft</td>
<td>14 ft</td>
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<tr>
<td>Existing Overpassing Bridge</td>
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<tr>
<td>Sign Truss / Pedestrian Bridge (22a)</td>
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<td>17.5 ft; Existing: 17 ft</td>
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<th>50 mph</th>
<th>55 mph</th>
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<tr>
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<td>200 ft</td>
<td>250 ft</td>
<td>360 ft</td>
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<td>495 ft</td>
</tr>
<tr>
<td>Stop Maneuver</td>
<td>490 ft</td>
<td>590 ft</td>
<td>800 ft</td>
<td>910 ft</td>
<td>1030 ft</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (28)</td>
<td>46-10.0</td>
<td>P: 355 ft</td>
<td>SUT: 450 ft</td>
<td>U: 415 ft</td>
<td>SUT: 525 ft</td>
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<tr>
<td>Minimum Radius for Emax = 4% / 6%</td>
<td>43-2.0</td>
<td>260 ft / 240 ft (24a)</td>
<td>420 ft / 390 ft (24a)</td>
<td>600 ft / 550 ft (24a)</td>
<td>750 ft (24b)</td>
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### Alignment Elements

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<th>HL-93</th>
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<td>45 mph</td>
<td>50 mph</td>
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<tr>
<td>Stop Maneuver</td>
<td>490 ft</td>
<td>590 ft</td>
<td>800 ft</td>
<td>910 ft</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (28)</td>
<td>46-10.0</td>
<td>P: 355 ft</td>
<td>SUT: 450 ft</td>
<td>U: 415 ft</td>
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<tr>
<td>Minimum Radius for Emax = 4% / 6%</td>
<td>43-2.0</td>
<td>260 ft / 240 ft (24a)</td>
<td>420 ft / 390 ft (24a)</td>
<td>600 ft / 550 ft (24a)</td>
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</tbody>
</table>

### Geometric Design Criteria for Urban Arterial, 4 or More Lanes

*Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO's A Policy on Geometric Design of Highways and Streets (the Green Book) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply to a route either on or off the National Highway System, regardless of funding source.
(1) **Design Speed.** The minimum design speed should equal the minimum value, the anticipated posted speed limit after construction, or the legal speed limit on a non-posted highway. The legal speed limit in an urban district is 30 mph. Based on an engineering study, the design speed may be raised to an absolute maximum of 55 mph.

(2) **On-Street Parking.** In general, on-street parking is discouraged.

(3) **Travel-Lane Width.** For an arterial on the National Truck Network, the right lane must be 12 ft in width.

(4) **Surface Type.** The pavement-type selection will be determined by the INDOT Office of Pavement Engineering.

(5) **Curb Offset.** The curb offset (for both left and right sides) should be 2 ft. Vertical curbs introduced intermittently should be offset 2 ft. A continuous curb used along a median or channelizing island may be offset 1 ft.

(6) **Shoulder Width.** The value applies to the paved shoulder width. The following will also apply.
   a. For an uncurbed section, the shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. For a curbed section, the curb offset is included in the paved shoulder width.

(7) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable for an existing bridge to remain in place.

(7A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

(8) **Curb Offset for Auxiliary Lane.** In a curbed section, the offset may be zero.

(9) **Parking Lane.** Where a parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be equal to the travel lane width plus a 1 ft offset to the curb (if present). The cross slope for a parking lane is typically 1% steeper than that of the adjacent travel lane.

(10) **Minimum Median Width.** The criteria assume the presence of a mountable curb with a 0 ft curb offset.

(11) **Sidewalk Width.** A buffer of less than 2 ft wide is not permitted. If no buffer is provided, the sidewalk width should be 6 ft.

(12) **Bicycle-Lane Width.** The value is in addition to the width of a parking lane, if present. See Section 51-7.0 for additional details.

(13) **Clear-Zone Width.** The following will apply.
   a. **Facility with Vertical Curbs.** The clear-zone width will be measured from the edge of travel lane or will be to the right-of-way line, whichever is less. No clear zone is required where there is 24-h parking.
   b. **Facility with Sloping Curbs or without Curbs.** The clear-zone width will vary according to design speed, traffic volume, side slopes, and horizontal curvature.
   c. **Curbed Facility.** There should be an appurtenance-free area as measured from the gutter line of a curb.
   d. **Value.** See Section 49-2.0 for specific clear-zone-width value.

(14) **Curbing Type.** Vertical curbs may only be used with design speed 45 mph or lower.
(15) **Side Slope, Uncurbed.** Value is for new construction. See Sections 45-3.0 and 45-8.0 for more information. For a reconstruction project, see Section 49-3.0.

(16) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(17) **Ditch Width.** A V-ditch should be used in a rock cut. See Section 45-8.0.

(18) **Backslope.** The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Sections 45-3.02 and 107-6.02 for typical rock-cut sections.

(19) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf will be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(20) **Structural Capacity, New or Reconstructed Bridge.** The following will apply.

   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.

(21) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:

   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(22) **Vertical Clearance, Arterial Under Railroad.** The following will apply.

   a. Value includes an additional 6 in. allowance for future pavement overlays.
   b. In a highly urbanized area, a minimum clearance of 14 ft may be provided if there is at least one route with a 16 ft clearance.
   c. Vertical clearance applies from usable edge to usable edge of shoulders.

(23) **Vertical Clearance, Arterial Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(24) **Minimum Radius.** The following will apply:

   a. Based on \( e_{\text{max}} = 4\% \) or 6\% and low-speed urban street conditions.
   b. Based on \( e_{\text{max}} = 8\% \) and open-road conditions.

(25) **Superelevation Rate.** See Section 43-3.0 for values of superelevation rate based on design speed and radius. See Section 43-3.0 and the INDOT *Standard Drawings* for information on superelevation requirements.

(26) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. Sometimes the stopping-sight-distance value for a truck will apply. See the discussion in Section 43-4.0.

(27) Where adjacent sidewalks are present, the maximum desirable grade is 5\%.

(28) **Intersection Sight Distance.** For a left turn onto a two-way, 4-lane undivided roadway: \( P = \) Passenger car; \( SUT = \) single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
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<td>Des: C; Min: C</td>
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<td>Uncurbed: Shoulder Width +4 ft</td>
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Des: Desirable; Min. Minimum.

* Level One controlling criterion, see page 2 of 4

GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, 2 LANES
(New Construction or Reconstruction)
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<th>Suburban</th>
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<tr>
<td>New or Reconstructed Bridge</td>
<td>Ch. 403</td>
<td>HL-93</td>
<td>HL-93</td>
<td>HL-93</td>
</tr>
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<td>*Clear-Roadway Width (20)</td>
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<td></td>
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<tr>
<td><strong>Existing Bridge to Remain in Place</strong></td>
<td>Ch. 72</td>
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</tr>
<tr>
<td>*Clear-Roadway Width</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Existing Overpassing Bridge</strong></td>
<td>Ch. 402-6.01</td>
<td>14 ft</td>
<td>14 ft</td>
<td>14 ft</td>
</tr>
<tr>
<td>New or Replaced Overpassing Bridge (21a)</td>
<td>44-4.0</td>
<td>16.5 ft</td>
<td>16.5 ft (21b)</td>
<td>16.5 ft (21b)</td>
</tr>
<tr>
<td>Existing Overpassing Bridge</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sign Truss / Pedestrian Bridge (21a)</td>
<td></td>
<td>14 ft</td>
<td>14 ft</td>
<td>14 ft</td>
</tr>
<tr>
<td>Vertical Clearance, Arterial Under (21)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Clearance, Arterial over Railroad (22)</td>
<td>Ch. 402-6.01</td>
<td>23 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Design Speed</strong></td>
<td></td>
<td>30 mph</td>
<td>35 mph</td>
<td>45 mph</td>
</tr>
<tr>
<td>*Stopping Sight Distance</td>
<td>42-1.0</td>
<td>200 ft</td>
<td>250 ft</td>
<td>300 ft</td>
</tr>
<tr>
<td>Stop Maneuver</td>
<td>490 ft</td>
<td>590 ft</td>
<td>800 ft</td>
<td>910 ft</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (27)</td>
<td>46-10.0</td>
<td>P: 330 ft</td>
<td>SUT: 420 ft</td>
<td>P: 390 ft</td>
</tr>
<tr>
<td>*Minimum Radius for $\epsilon_{\text{max}} = 4% / 6%$</td>
<td>43-2.0</td>
<td>260 ft/240 ft (23 a)</td>
<td>420 ft/390 ft (23a)</td>
<td>600 ft/550 ft (23a)</td>
</tr>
<tr>
<td>*Superelevation Rate (24)</td>
<td>43-3.0</td>
<td>Up to $\epsilon_{\text{max}}=6%$</td>
<td>$\epsilon_{\text{max}}=8%$</td>
<td></td>
</tr>
<tr>
<td><strong>Alignment Elements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*Horizontal Sight Distance</td>
<td>43-4.0</td>
<td>(25)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>*Vertical Curvature, K-value</td>
<td></td>
<td></td>
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<tr>
<td>Crest</td>
<td>44-3.0</td>
<td>19</td>
<td>29</td>
<td>61</td>
</tr>
<tr>
<td>Sag</td>
<td>37</td>
<td>49</td>
<td>79</td>
<td>96</td>
</tr>
<tr>
<td>*Maximum Grade (26)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Level</td>
<td>44-1.02</td>
<td>8%</td>
<td>7%</td>
<td>6.5%</td>
</tr>
<tr>
<td>Rolling</td>
<td>9%</td>
<td>8%</td>
<td>7.5%</td>
<td>7%</td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>Desirable: 0.5% Minimum: 0.3% (Curbed) 0.0% (Uncurbed)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

U: Urban; SU: Suburban.

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO's *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply to a route on or off the National Highway System, regardless of funding source.
1. **Design Speed.** The minimum design speed should equal the minimum value, the anticipated posted speed limit after construction or the legal speed limit on a non-posted highway. The legal speed limit in an urban district is 30 mph. Based upon an engineering study, the design speed may be raised to an absolute maximum of 55 mph.

2. **On-Street Parking.** In general, on-street parking is discouraged.

3. **Travel-Lane Width.** For an arterial on the National Truck Network, lane widths must be 12 ft.

4. **Surface Type.** The pavement-type selection will be determined by the INDOT Office of Pavement Engineering.

5. **Curb Offset.** The curb offset should be 2 ft. Vertical curbs introduced intermittently should be offset 2 ft. A continuous curb used along a median or channelizing island may be offset 1 ft.

6. **Shoulder Width.** The value applies to the paved-shoulder width. The following will also apply.
   a. For an uncurbed section, the shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. For a curbed section, the curb offset is included in the paved shoulder width.

7. **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

7A. **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

8. **Curb Offset for Auxiliary Lane.** In a curbed section, the offset may be zero.

9. **Parking Lane.** Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be equal to the travel lane width plus a 1 ft offset to the curb (if present). The cross slope for a parking lane is typically 1% steeper than that of the adjacent travel lane.

10. **Sidewalk Width.** A buffer of less than 2 ft wide is not permitted. If no buffer is provided, the sidewalk width should be 6 ft.

11. **Bicycle-Lane Width.** The value is in addition to the width of a parking lane, if present. See Section 51-7.0 for additional details.

12. **Clear-Zone Width.** The following will apply.
   a. **Facility with Vertical Curbs.** The clear-zone width will be measured from the edge of travel lane or will be to the right-of-way line, whichever is less. No clear zone is required where there is 24-h parking.
   b. **Facility with Sloping Curbs or without Curbs.** The clear-zone width will vary according to design speed, traffic volume, side slopes, and horizontal curvature.
   c. **Curbed Facility.** There should be an appurtenance-free area as measured from the gutter line of a curb.
   d. **Value.** See Section 49-2.0 for specific clear-zone-width value.

13. **Curbing Type.** Vertical curbs may only be used with design speed 45 mph or lower.
(14) **Side Slope, Uncurbed.** Value is for new construction. See Section 45-3.0 for more information. For a reconstruction project, see Section 49-3.0.

(15) **Foreslope.** See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(16) **Ditch Width.** A V-ditch should be used in a rock cut.

(17) **Backslope.** The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Sections 45-3.02 and 107-6.02 for typical rock-cut sections.

(18) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf will be 6 ft. Where a sidewalk is present, the toe of the backslope will be 2 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(19) **Structural Capacity, New or Reconstructed Bridge.** The following will apply.
   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.

(20) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and

(21) **Vertical Clearance, Arterial Under Railroad.** The following will apply.
   a. Value includes an additional 6 in. allowance for future pavement overlays.
   b. In a highly urbanized area, a minimum clearance of 14 ft may be provided if there is at least one route with a 16-ft clearance.
   c. Vertical clearance applies from usable edge to usable edge of shoulder.

(22) **Vertical Clearance, Arterial Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(23) **Minimum Radius.** The following will apply:
   a. Based on $e_{\text{max}} = 4\%$ or 6\% and low-speed urban street conditions.
   b. Based on $e_{\text{max}} = 8\%$ and open-road conditions.

(24) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius. See Section 43-3.0 and the INDOT Standard Drawings for information on superelevation requirements.

(25) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. Sometimes the stopping-sight-distance value for a truck will apply. See the discussion in Section 43-4.0.

(26) Where adjacent sidewalks are present, the maximum desirable grade is 5\%.

(27) **Intersection Sight Distance.** For a left turn onto a 2-lane roadway: P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.

**GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, 2 LANES**
*(New Construction or Reconstruction)*

**Figure 53-7 (Page 4 of 4)**
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Design Value (By Type of Area)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Controls</td>
<td></td>
<td>Suburban</td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>40-2.02</td>
<td>20 Years</td>
</tr>
<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Desirable: C; Minimum: D</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>45-1.04</td>
<td>Optional (3)</td>
</tr>
<tr>
<td>Travel Lane</td>
<td></td>
<td>Curbed: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>*Width (4)</td>
<td>45-1.01</td>
<td>Uncurbed: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>Typical Surface Type (5)</td>
<td></td>
<td>Curbed: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>Curved</td>
<td>45-1.02</td>
<td>Uncurbed: 8 ft; Min: 2 ft</td>
</tr>
<tr>
<td>*Curb Offset (6)</td>
<td></td>
<td>Curved: Des: 8 ft; Min: 2 ft</td>
</tr>
<tr>
<td>Shoulder</td>
<td></td>
<td>Uncurbed: 8 ft; Min: 2 ft</td>
</tr>
<tr>
<td>8 ft</td>
<td></td>
<td>8 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>45-1.02</td>
<td>Des: 8 ft; Min: 2 ft</td>
</tr>
<tr>
<td>Lane Width</td>
<td>45-1.03</td>
<td>Curved: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>Curb Offset</td>
<td></td>
<td>Uncurved: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td></td>
<td>Curved: Des: 12 ft; Min: 11 ft</td>
</tr>
<tr>
<td>Shoulder (8A)</td>
<td></td>
<td>Uncurved: 11 ft; Min: 8 ft</td>
</tr>
<tr>
<td>Typical Surface Type (5)</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>TWLTL Width</td>
<td>46-5.0</td>
<td>Des: 14 ft; Min: 12 ft</td>
</tr>
<tr>
<td>Parking-Lane Width (1)</td>
<td>45-1.04</td>
<td>Des: 11 ft; Min: 8 ft</td>
</tr>
<tr>
<td>Median Width</td>
<td>45-2.0</td>
<td>Des: 16 ft; Min: 4 ft (9)</td>
</tr>
<tr>
<td>Raised Island</td>
<td>45-2.0</td>
<td>Des: 16 ft; Min: 4 ft (9)</td>
</tr>
<tr>
<td>Flush / Corrugated</td>
<td></td>
<td>Des: 16 ft; Min: 4 ft (9)</td>
</tr>
<tr>
<td>Sidewalk Width (10)</td>
<td>45-1.06</td>
<td>5 ft with 5 ft Buffer (Des)</td>
</tr>
<tr>
<td>Bicycle-Lane Width (11)</td>
<td>51-7.0</td>
<td>Curbed: 5 ft</td>
</tr>
<tr>
<td>Clear-Zone Width</td>
<td>49-2.0</td>
<td>Curbed: Shld. Width +4 ft</td>
</tr>
<tr>
<td>Typical Curbing Type, where used (13)</td>
<td>45-1.05</td>
<td>Sloping / Vertical</td>
</tr>
<tr>
<td>Cut</td>
<td>45-3.0</td>
<td>Sloping / Vertical</td>
</tr>
<tr>
<td>Ditch Width</td>
<td>45-3.0</td>
<td>Sloping / Vertical</td>
</tr>
<tr>
<td>Foreslope</td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>4 ft (16)</td>
<td></td>
<td>4:1 for 4 ft; 3:1 Max. to Top (17)</td>
</tr>
<tr>
<td>Backslope</td>
<td></td>
<td>4:1 for 4 ft; 3:1 Max. to Top (17)</td>
</tr>
<tr>
<td>Fill</td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>Des: 6:1 to Cir Zone; 3:1 Max to Toe</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:1 for 4 ft; 3:1 Max. to Top (17)</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Side Slopes, Uncurbed (14)</td>
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<td>N/A</td>
</tr>
<tr>
<td>Cut (Backslope)</td>
<td>45-3.0</td>
<td>Des: 6:1 to Cir Zone; 3:1 Max to Toe</td>
</tr>
<tr>
<td>12:1 for 12 ft; 3:1 Max to Toe</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fill (19)</td>
<td></td>
<td>N/A</td>
</tr>
</tbody>
</table>

Des: Desirable  Min: Minimum  
U: Urban  SU: Suburban

* Level One controlling criterion, see page 2 of 4

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR
(New Construction or Reconstruction)
Figure 53-8 (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Design Value (By Type of Area)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bridges</strong></td>
<td>Suburban</td>
</tr>
<tr>
<td><strong>New or Reconstructed Bridge</strong></td>
<td>Ch. 403</td>
</tr>
<tr>
<td><em>Structural Capacity (20)</em></td>
<td></td>
</tr>
<tr>
<td>45-4.01</td>
<td>HL-93</td>
</tr>
<tr>
<td><em>Clear-Roadway Width (21)</em></td>
<td></td>
</tr>
<tr>
<td>14.5 ft</td>
<td>14.5 ft</td>
</tr>
<tr>
<td><strong>Existing Bridge to Remain in Place</strong></td>
<td>Ch. 72</td>
</tr>
<tr>
<td><em>Structural Capacity</em></td>
<td></td>
</tr>
<tr>
<td>45-4.01</td>
<td>HS-20</td>
</tr>
<tr>
<td><em>Clear-Roadway Width</em></td>
<td></td>
</tr>
<tr>
<td>14 ft</td>
<td>14 ft</td>
</tr>
<tr>
<td><strong>Vertical Clearance, Collector under (22)</strong></td>
<td>New or Replaced Overpassing Bridge</td>
</tr>
<tr>
<td>Existing Overpassing Bridge</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Clearance, Collector over Railroad (23)</strong></td>
<td>Ch. 402-6.01</td>
</tr>
<tr>
<td><strong>Alignment Element</strong></td>
<td>Design Speed</td>
</tr>
<tr>
<td><em>Stopping Sight Distance</em></td>
<td>42-1.0</td>
</tr>
<tr>
<td>Decision Sight Distance</td>
<td>42-2.0</td>
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<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stop Maneuver</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (28)</td>
<td>46-10.0</td>
</tr>
<tr>
<td></td>
<td>SUT: 420 ft</td>
</tr>
<tr>
<td>*Minimum Radius for $e_{max} = 4% / 6%$ (24a)</td>
<td>43-2.0</td>
</tr>
<tr>
<td><em>Superelevation Rate (25)</em></td>
<td>43-3.0</td>
</tr>
<tr>
<td><em>Horizontal Sight Distance</em></td>
<td>43-4.0</td>
</tr>
<tr>
<td><strong>Vertical Curvature, K-value</strong></td>
<td>Crest</td>
</tr>
<tr>
<td></td>
<td>Sag</td>
</tr>
<tr>
<td><strong>Maximum Grade (27)</strong></td>
<td>Level</td>
</tr>
<tr>
<td></td>
<td>Rolling</td>
</tr>
</tbody>
</table>

U: Urban SU: Suburban

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.

These criteria apply regardless of funding source.

**GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR**

(New Construction or Reconstruction)

*Figure 53-8 (Page 2 of 4)*
(1) **Parking Lane.** In a residential area, a parallel parking lane of 7 to 8 ft width should be provided on one or both sides of the street. In a commercial or industrial area, parking-lane width should range from 8 to 11 ft, and lanes should usually be provided on both sides of the street. The minimum value may only be used if the lane is not intended for use as a travel lane in a restricted condition. Where a curb-and-gutter section is used, the gutter-pan width may be considered as part of the parking-lane width. Where practical, the parking-lane width should be in addition to the gutter-pan width.

(2) **Design Speed.** The minimum design speed should equal the minimum value, the anticipated posted speed limit after construction, or the legal speed limit on a non-posted highway. The legal speed limit in an urban district is 30 mph. Based upon an engineering study, the design speed may be raised to an absolute maximum of 55 mph.

(3) **On-Street Parking.** In general, on-street parking is discouraged.

(4) **Travel-Lane Width.** In an industrial area, a 12 ft width should be used. Where right-of-way is restricted, an 11 ft width may be used in an industrial area, or a 10 ft width may be used in a residential area. On a multi-lane facility in a built-up area, the minimum width is 10 ft.

(5) **Surface Type.** The pavement-type selection will be determined by the INDOT Office of Pavement Engineering.

(6) **Curb Offset.** The curb offset should be 2 ft. Vertical curbs introduced intermittently should be offset 2 ft. A continuous curb used along a median or channelizing island may be offset 1 ft.

(7) **Shoulder Width.** The value applies to paved-shoulder width. The following will also apply.
   a. For an uncurbed section, the shoulder is paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. For a curbed section, the curb offset is included in the paved shoulder width.

(8) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(8A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.

(9) **Minimum Median Width.** The criteria assume the presence of mountable curbs with a 0 ft curb offset.

(10) **Sidewalk Width.** A buffer of less than 2 ft wide is not permitted. If no buffer is provided, the sidewalk width should be 6 ft.

(11) **Bicycle-Lane Width.** The width is in addition to the width of a parking lane, if present. See Section 51-7.0 for additional details.

(12) **Clear-Zone Width.** The following will apply.
   a. Facility with Vertical Curbs. The clear-zone width will be measured from the edge of travel lane or will be to the right-of-way line, whichever is less. No clear zone is required where there is 24-h parking.
   b. Facility with Sloping Curbs or without Curbs. The clear-zone width will vary according to design speed, traffic volume, side slopes, and horizontal curvature.
   c. Curbed Facility. There should be an appurtenance-free area as measured from the gutter line of a curb.
   d. Value. See Section 49-2.0 for specific clear-zone-width value

(13) **Curbing Type.** Vertical curbs may only be used with a design speed 45 mph or lower.
(14) Side Slopes, Uncurbed. Value is for new construction. See Sections 45-3.0 and 45-8.0 for more information. For a reconstruction project, see Section 49-3.0.

(15) Foreslope. See Sections 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(16) Ditch Width. A V-ditch should be used in a rock cut.

(17) Backslope. The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See Section 45-3.02 and 107-6.02 for typical rock-cut sections.

(18) Side Slope, Curbed, Cut. A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf will be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(19) Side Slope, Curbed, Fill. If no sidewalks are present or planned, the lateral extent of the 12:1 slope may be reduced to 4 ft.

(20) Structural Capacity, New or Reconstructed Bridge. The following will apply.
   a. A State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. A bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.
   c. See Chapter 403 for additional information on the loading configurations.

(21) Width, New or Reconstructed Bridge. See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(22) Vertical Clearance, Collector Under. Value includes an additional 6 in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulder.

(23) Vertical Clearance, Collector Over Railroad. See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(24) Minimum Radius. The following will apply.
   a. Based on \( e_{\text{max}} = 4\% \) or 6\% and low-speed urban street conditions.
   b. Based on \( e_{\text{max}} = 8\% \) and open-road conditions.

(25) Superelevation Rate. See Section 43-3.0 for value of superelevation rate based on design speed and radius. See Section 43-3.0 and the INDOT Standard Drawings for information on superelevation requirements.

(26) Horizontal Sight Distance. For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. See the discussion in Section 43-4.0.

(27) Maximum Grade. For a grade along a longitudinal distance of less than 500 ft (PVT to PVC), a one-way downgrade, or a road with AADT < 400, the maximum grade may be up to 2\% steeper than the table value. Where adjacent sidewalks are present, the maximum desirable grade is 5\%.

(28) Intersection Sight Distance. For a left turn onto a 2-lane roadway: \( P = \) Passenger car; \( \text{SUT} = \) single unit truck. See Figure 46-10G for value for a combination truck.

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR
(New Construction or Reconstruction)

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<table>
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Des: Desirable Min: Minimum
U: Urban   SU: Suburban

* Level One controlling criterion, see page 2 of 4

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### GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREET

(New Construction or Reconstruction)

Figure 53-9 (Page 2 of 4)
(1) **Parking Lane.** In a residential area, the minimum width is 7 ft. In a commercial or industrial area the minimum width is 8 ft. Where curb-and-gutter sections are used, the gutter width should be considered part of the parking lane width.

(2) **Design Speed.** The minimum design speed should equal the minimum value, the anticipated posted speed limit after construction, or the legal speed limit on a non-posted highway. The legal speed limit in an urban district is 30 mph. Based upon an engineering study, the design speed may be raised to an absolute maximum of 55 mph.

(3) **On-Street Parking.** In general, on-street parking is discouraged.

(4) **Travel-Lane Width.** In a restricted area and where there are few trucks, a width of 1 ft narrower than the value may be used, but the total width may not be less than 10 ft. In an industrial area, a 12 ft width should be used. In a residential area, a 26 ft roadway (curb face to curb face) consisting of one 12 ft travel lane and two 7 ft parking lanes is used. In an industrial area, a 12 ft width is desirable and an 11 ft width is minimum.

(5) **Curb Offset.** The curb offset should be 2 ft. For a curbed section, the curb offset is included in the paved-shoulder width.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(7) **Sidewalk Width.** A buffer of less than 2 ft wide is not permitted. If no buffer is provided, the sidewalk width should be 6 ft.

(8) **Bicycle-Lane Width.** The value is in addition to the width of a parking lane, if present. See Section 51-7.0 for additional details.

(9) **Clear-Zone Width.** The following will apply.
   a. **Facility with Vertical Curbs.** The clear-zone width will be measured from the edge of travel lane or will be to the right-of-way line, whichever is less. No clear zone is required where there is 24-h parking.
   b. **Facility with Sloping Curbs or without Curbs.** The clear-zone width will vary according to design speed, traffic volume, side slopes, and horizontal curvature.
   c. **Curbed Facility.** There should be an appurtenance-free area as measured from the gutter line of a curb. Vertical curbs may only be used with design speed 45 mph or lower.
   d. **Value.** See Section 49-2.0 for specific clear-zone-width values.

(10) **Backslope.** The backslope for a rock cut will vary according to the height of the cut and the geotechnical requirements. See the INDOT Standard Drawings for typical rock-cut sections.

(11) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf is 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

---

**GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREET**  
(New Construction or Reconstruction)  
Figure 53-9 (Page 3 of 4)
(12) **Side Slope, Curbed, Fill.** If no sidewalks are present or planned, the lateral extent of the 12:1 slope may be reduced to 4 ft.

(13) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(14) **Width, Existing Bridge to Remain in Place.** If the width of an existing bridge is less than the approach travelway width, consideration should be given to widening the bridge. For such a bridge of length greater than 200 ft, the minimum shoulder width on the right and the left sides is 3.5 ft.

(15) **Vertical Clearance, Local Street Under.** Value includes an additional 6-in. allowance for future pavement overlays. Vertical clearance applies from usable edge to usable edge of shoulder.

(16) **Vertical Clearance, Local Street Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(17) **Minimum Radius.** This is based on $e_{\text{max}}=4\%$ and low-speed urban street conditions.

(18) **Superelevation Rate.** See Section 43-3.0 for value of superelevation rate based on design speed and radius. See Section 43-3.0 for information on superelevation requirements.

(19) **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the radius and the sight distance which applies at the site. See the discussion in Section 43-4.0.

(20) **Maximum Grade.** In a residential area, the maximum grade should not exceed 15%. In an industrial or commercial area, the maximum grade should not exceed 8%.

(21) **Flat Terrain.** In very flat terrain and where no drainage outlet is available, a gutter grade as low as 0.2% may be used.

(22) **Intersection Sight Distance.** For a left turn onto a 2-lane roadway: P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
CHAPTER 54

Geometric Design of Existing Freeway (3R) or (4R) Partial Reconstruction

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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CHAPTER 54

GEOMETRIC DESIGN OF EXISTING
FREEWAY (3R) OR (4R)
PARTIAL RECONSTRUCTION

54-1.0 GENERAL

54-1.01 Background

The Department began construction of its freeway system in the 1950s, and today the Indiana system has been completed. The freeway system has introduced a level of mobility and safety for the traveling public which was unattainable without its special features, such as full control of access, wide roadway widths, and higher design speeds.

The freeway system requires periodic repair and upgrading which exceeds the limits of normal maintenance. Such a capital improvement is defined as a 3R project (resurfacing, restoration, and rehabilitation), partial-reconstruction (4R) project, or full-reconstruction (4R) project. This Chapter discusses the Department’s design criteria for a 3R or partial 4R reconstruction project on a freeway. These criteria meet or exceed the criteria described in AASHTO’s A Policy on Design Standards - Interstate System and AASHTO’s A Policy on Geometric Design of Highways and Streets. A full-reconstruction project should be designed in accordance with the criteria described elsewhere in this Manual.

54-1.02 Applicability

54-1.02(01) Freeway Definition

Within the functional-classification system, a freeway is the highest level of arterial. Such a facility is characterized by full control of access, divided roadways, high design speed, and a high level of driver comfort and safety. Each Interstate highway as well as any other route with full control of access is classified as a freeway (e.g., US 31 around South Bend, SR 912 in Lake County, Airport Expressway in Indianapolis). See Section 40-1.0 for more information on the functional-classification system and the role of the freeway within the system.
54-1.02(02) Project Scope of Work

Section 40-6.01 defines the typical types of improvements that are made on a 3R or reconstruction project on the National Highway System (NHS). The following provides an overview of what may represent a 3R freeway project or a freeway reconstruction project. For a more in-depth description, the designer should review Section 40-6.01. For a freeway, the distinction between 3R, partial reconstruction, and complete reconstruction can be summarized as follows:

1. **3R Project.** A 3R freeway project may include the improvements as follows:
   
   a. pavement resurfacing;
   
   b. full-depth pavement reconstruction, if the reconstructed pavement area is 30% or less of the traveled way;
   
   c. widening existing travel lanes or shoulders;
   
   d. upgrading the structural strength of shoulders;
   
   e. improving the superelevation of existing horizontal curves;
   
   f. adding auxiliary lanes;
   
   g. improving roadway delineation;
   
   h. upgrading roadside safety;
   
   i. increasing the length of acceleration and deceleration lanes at an interchange;
   
   j. widening an existing bridge as part of a bridge reconstruction project;
   
   k. upgrading or replacing bridge railings;
   
   l. overlaying bridge decks;
   
   m. preservation of bridge substructures;
   
   n. improving roadside drainage;
   
   o. widening existing ramps;
p. flattening horizontal or vertical curves; or
q. increasing the vertical clearance at underpasses.

2. **Partial-Reconstruction (4R) Project.** A partial-reconstruction (4R) freeway project may include the improvements as follows:

   a. more than 30% of the travelway pavement area must be removed and replaced,
   
   b. a concrete overlay of a least 6 in. is required, or an asphalt overlay of at least 8 in. is to be placed;
   
   c. the facility cannot adequately accommodate the current or projected (10-year) traffic demand and additional lanes are necessary;
   
   d. major revisions are necessary to the existing horizontal and vertical alignment requiring that more than 30% of the travelway pavement must be replaced;
   
   e. total bridge or bridge-deck replacement is required;
   
   f. bridge-deck widening is necessary due to added travel lanes on the approaches; or
   
   g. interchange upgrading is required to meet current and projected (20-year) traffic demands.

3. **Complete-Reconstruction (4R) Project.** A freeway improvement is considered to be a complete reconstruction if the project intent is to replace the existing facility. Complete reconstruction will typically provide significant improvements in level of service, operational efficiency, and safety. For a complete-reconstruction project, the criteria described in Chapter 53 should be used.

**54-1.03 Objectives**

The basic objective of a 3R/partial 4R freeway project is to improve the freeway’s serviceability to meet future demands by extending the service life of the existing facility and enhancing highway safety. This objective applies to all aspects of the freeway’s serviceability. If a project is classified as a partial 4R project, an additional objective, where practical, is to upgrade existing elements to
new-construction criteria. For example, where the pavement is to be replaced, it may be practical to improve the horizontal or vertical alignment.

54-1.04 Approach

A 3R/Partial 4R freeway project is most-often initiated to make a specific improvement to the freeway (e.g., resurfacing or roadside-safety improvements). The Department’s policy is to review and upgrade other design elements, wherever practical. The Department’s 3R/partial 4R approach is summarized as follows.

1. **Nature of Improvements.** Identify the specific improvements intended for the project. The designer should review Section 54-1.02(02) for typical freeway-project improvements.

2. **Numerical Criteria.** The criteria are based on AASHTO *A Policy on Design Standards - Interstate System* and the AASHTO *Policy on Geometric Design of Highways and Streets*, new construction/reconstruction criteria for a freeway. Sections 54-2.0 through 54-6.0 provide the 3R/partial 4R freeway criteria. Unless stated in this chapter, the freeway-design criteria described elsewhere in this *Manual* should be incorporated where practical.

3. **Secondary Impact.** Identify and evaluate any secondary impact which may be precipitated due to the freeway improvement. Examples are as follows:
   a. the installation of a median barrier may restrict horizontal sight distance;
   b. a pavement overlay may reduce the vertical clearance requirements under a bridge;
   c. a pavement overlay may require the adjustment of roadside-barrier height.

4. **Other Improvements.** Identify geometric design deficiencies within the project limits which can be practically corrected without exceeding the intended project scope of work. A review of the accident history is important in conducting this evaluation.

5. **Design Exception.** The discussion in Section 40-8.0 on design exceptions applies to the geometric design of a 3R/partial 4R freeway project. However, the designer should evaluate the proposed design against the criteria described in this chapter. The need for a design exception should be based on the minimum AASHTO Interstate System criteria that were in effect at the time of original construction or when the facility was incorporated into the Interstate system. These design elements include the following:
   a. horizontal alignment, except superelevation;
   b. vertical alignment;
c. shoulder widths; and  
d. median width.

54-1.05 3R or Partial 4R Project Evaluation

Sections 54-2.0 through 54-6.0 provide the specific geometric design and roadside-safety criteria which will be used to determine the design of a 3R/partial 4R freeway project. The following should also be evaluated as described below.

1. **Accident Experience.** The historical accident data within the project limits should be evaluated. Accident data is available from the Office of Environmental Services. Section 55-8.0 further describes the Department’s accident-analysis procedure.

2. **Existing Geometrics.** The designer will review the as-built plans and combine this review with the field review and field survey (if conducted) to determine the existing geometrics within the project limits. This includes lane and shoulder widths, horizontal and vertical alignment, interchange geometrics, and roadside-safety design.

3. **Physical Constraints.** The physical constraints within the project limits will often determine what geometric improvements are practical and cost-effective. These include topography, adjacent development, available right of way, utilities, or environmental constraints (e.g., wetlands).

4. **Field Review.** The designer will conduct a thorough field review of the proposed project. Other personnel should attend the field review as appropriate, including personnel from the district traffic, maintenance, and construction offices. The objective of the field review should be to identify potential safety hazards and potential safety improvements to the facility.

5. **Pavement Condition.** A 3R/Partial 4R project is programmed because of a significant deterioration of the existing pavement structure. The extent of deterioration will determine the necessary level of pavement improvements, which may include milling of the existing pavement surface or replacement of the pavement. This decision will also influence the extent of practical geometric improvements. For a freeway to be eligible for pavement resurfacing or replacement, the pavement should exhibit one or more of the conditions as follows:

   a. alligator cracking;  
   b. bleeding;
c. block (cracking);  
d. bump (upheaval);  
e. corrugation;  
f. depression and rutting;  
g. edge cracking;  
h. longitudinal or transverse cracking;  
i. patching or utility cut;  
j. polished aggregate;  
k. potholing;  
l. slippage-cracking; or  
m. weathering and raveling.

Pavement resurfacing or replacement will be based upon the design-year traffic data, at 10 years for resurfacing or 20 years for reconstruction. The pavement surface should be designed to incorporate skid resistance.

6. **Geometric Design of Adjacent Highway Sections.** The designer should examine the geometric features and operating speeds of the freeway sections adjacent to the project. This will include investigating whether or not highway improvements are in the planning stages. The project should provide design continuity with the adjacent sections. This involves a consideration of factors such as driver expectancy, geometric design consistency, and proper transitions between sections with different geometric designs.

7. **Early Coordination for Right-of-Way Acquisition or Utilities Coordination.** Significant right-of-way acquisitions are typically outside the scope of a 3R/partial 4R freeway project. However, the field review and accident or speed studies may indicate the need for selective safety improvements or other minor operational improvements which will require right of way purchases (e.g., interchange improvements). Therefore, the designer should, as early as feasible, determine the improvements which will be incorporated into the project design and initiate the right-of-way acquisition process.

8. **Maintenance and Protection of Traffic.** For work on an existing alignment, maintenance and protection of traffic during construction should be considered in project development. The protection of construction workers should also be considered. The designer should see Part VIII for criteria on the design of a work zone for traffic accommodation.

9. **Traffic-Control Devices.** Signing and pavement markings should be in accordance with Part VII and the *Manual on Uniform Traffic Control Devices* (MUTCD). The Highway Operations Division’s Office of Traffic Engineering is responsible for selecting, locating, and analyzing the adequacy of breakaway or yielding sign or light supports. However, the
designer should work with the Office of Traffic Engineering to identify possible geometric and safety deficiencies which will remain in place (i.e., no improvement will be made). The Office of Traffic Engineering will then determine if additional signing, traffic-control devices, or delineation treatments are warranted.

10. **Documenting the Design Process.** The Office of Environmental Services will prepare the Engineer’s Report which will address the following:

   a. existing geometric and roadside features, traffic volumes and speeds, and accident history;

   b. applicable minimum design criteria;

   c. specific safety problems or concerns raised as a result of a review of accident data, by a field inspection, or by the public;

   d. design options for correcting safety problems and the cost, safety, and other relevant impacts of these options;

   e. proposed exceptions to applicable design criteria and the rationale to support the exceptions; and

   f. the recommended design proposal.

The Office of Environmental Services will identify design exceptions that will be required. The designer will be responsible for the preparation of a design-exception request (See Section 40-8.0).

**54-2.0 TABLE OF 3R OR PARTIAL 4R FREEWAY GEOMETRIC-DESIGN VALUES**

Figure 54-2A provides the Department’s criteria for the design of a 3R or partial 4R freeway project for either a rural or an urban area. The designer should consider the following in the use of the table.

1. **Design Manual Section References.** The designer should review the appropriate section references for greater insight into the design elements.

2. **Footnotes.** The table includes footnotes which are identified by a number in parentheses, e.g., (6). The information in the footnotes is critical to the proper use of the table.
3. **Controlling Design Criteria.** Controlling design criteria are identified with an asterisk. The designer should evaluate the proposed design against the criteria shown in the table and elsewhere in this chapter.

4. **Design Exception.** These standards are for use on an existing freeway including that on the National Highway System. They are to be used for each project that is classified as 3R or partial reconstruction regardless of funding source. Deviation from controlling design criteria should be addressed in an approved design exception. Operational or maintenance changes, permanent or temporary, exclusive of work-zone traffic control that create substandard conditions such as by re-striping to obtain added lane(s) by reducing existing lane widths or shoulders, must be addressed in a design exception whether or not actual construction or reconstruction is involved.

### 54-3.0 GEOMETRIC DESIGN

Though Figure 54-2A provides the required geometric-design criteria, the designer must still make certain decisions, such that some flexibility can be applied. These are discussed below.

The design criteria used for horizontal alignment excluding superelevation, vertical alignment, and width of median or shoulders may be the AASHTO Interstate System criteria that were in effect at the time of the route’s original construction or inclusion into the Interstate System.

### 54-3.01 Design Controls

#### 54-3.01(01) Traffic-Volume Analysis

1. **Design Life.** The pavement-resurfacing portion of a 3R project should be designed using a 10-year design life. All other elements should have a design life of 20 years beyond the expected construction date.

2. **Level of Service (LOS).** Figure 54-2A provides the desirable and minimum LOS criteria. The geometric-design elements should be designed to be in accordance with the level-of-service criteria for a design hourly volume at 20 years beyond the expected completion date.

3. **Traffic Data.** The designer should obtain the necessary traffic data from the Office of Environmental Services. This should include current and future (10 and 20 years) AADT, DHV, percent of trucks and buses (including that for each interchange), accident data for the most recent 3-year period, and any known future traffic impact.
4. **Capacity Analysis.** The analytical techniques in the *Highway Capacity Manual* and Chapter 41 will be used to conduct the capacity analysis.

**54-3.01(02) Design Speed**

Chapter 53 provides the Department’s criteria for selecting the design speed for a new construction or complete 4R freeway project. These will also apply to a 3R/partial 4R freeway project. As a minimum, the design speed for the original work may be used. Under restricted urban conditions, the existing posted speed limit may be used as the design speed.

The design speed selected must equal or exceed the existing posted speed limit or a design exception will be required. See Section 40-4.0 for additional information on design speed.

**54-3.02 Horizontal and Vertical Alignment**

Unless the specific objective of the freeway project is to improve one or more horizontal- or vertical-alignment features, the existing alignment will be acceptable under the conditions as follows:

1. the design is in accordance with the AASHTO Interstate System criteria that were in effect at the time of the route’s original construction or inclusion into the Interstate system; and

2. a review of the accident history for the past three years does not indicate a problem.

Once the decision has been made to reconstruct a horizontal- or vertical-alignment feature, the designer should apply the criteria described in Chapter 43 or 44.

**54-3.02(01) Superelevation**

On a horizontal curve where the existing radius will be retained, it may be necessary to make improvements to the superelevation. This may require revising the pavement-resurfacing thickness to meet the superelevation criteria described in Sections 43-2.0 and 43-3.0. Where the pavement structure will be reconstructed, the superelevation design should be in accordance with the new construction criteria described in Sections 43-2.0 and 43-3.0.
54-3.02(02) Grades

The maximum grades are shown in Figure 54-2A.

54-3.02(03) Vertical Clearance

The minimum vertical clearance is 16 ft over the entire roadway including the usable shoulder widths for both the left and right shoulders. If practical, the 16-ft clearance should be provided at each overpass within the project limits. If the 16-ft clearance cannot be obtained, a design exception will be required. However, for the routes in Marion County listed below, an existing overpass with a vertical clearance of at least 14 ft may be retained without a design exception.

1. I-65 from I-465 South to I-465 North;
2. I-70 from I-465 East to I-465 West; and

A low-clearance warning sign should be provided for each structure with a vertical clearance of less than 14.5 ft.

54-3.03 Cross Section

54-3.03(01) Lane and Shoulder Width

Each travel-lane or shoulder width not in accordance with Figure 54-2A should be evaluated for widening.

1. Travel Lane. The width of each travel lane or auxiliary lane should be 12 ft.

2. Shoulder. Existing shoulder widths may be retained if they are in accordance with the AASHTO Interstate System criteria in effect at the time of the route’s original construction or inclusion into the Interstate system.

54-3.03(02) Curbs

1. Safety Considerations. All existing curbs should be removed for safety reasons, unless they are required for drainage.
2. **Type.** If curbing is required for drainage, only sloping curbs will be permitted.

3. **Guardrail.** A curb in front of a guardrail may cause an errant vehicle to vault over or break through the barrier. Where guardrail is used and curbing is necessary for drainage, the maximum curb height should be 4 in. and should be placed behind the front face of the guardrail.

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54-3.03 Median

1. **Width.** The existing width should be retained.

2. **Parallel Slopes.** Existing slopes of 4:1 or flatter should be retained. If existing slopes are flattened, the designer should consider the effect on drainage within the median.

3. **Transverse Slopes.** Transverse slopes for ditch checks or median crossovers should be 10:1 or flatter.

4. **Median Opening.** See Section 54-6.0 for information.

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54-3.03(04) Fill or Cut Slope

1. **No Roadway Widening.** An existing slope of 2:1 or flatter should be retained. However, a slope steeper than 4:1 should be evaluated for flattening.

2. **Roadway Widening.** If the lanes or shoulders are widened as part of the project, this will produce a steeper fill slope or ditch foreslope (assuming the toe of fill slope or toe of backslope remains in the same location). The roadside design should be modified to provide a configuration which is the same as or flatter than the roadside cross section before the project limits. As a minimum, the following will apply.

   a. **Embankment slope.** A fill slope or ditch foreslope beginning at the shoulder break should not be steeper than 4:1 unless steeper slopes can be justified in an engineering and economic analysis. If the slope can be made flatter than 4:1, the designer should desirably provide a 6:1 slope at least within the clear zone.

   b. **Ditch.** If right of way is available, the existing ditch line should be moved outward and the slopes flattened as much as practical. A drainage ditch within the clear zone
should be regraded as much as practical to make it traversable for an errant vehicle. See Section 49-3.02 for information on a traversable ditch.

c. Embankment Stability. Stable embankment material is required. Sod or other appropriate materials or methods should be provided where erosion may be considered a problem.

3. Roadside Safety. Upgrading the roadside safety is often an objective of the project. The designer should consider the safety benefits of flattening fill or cut slopes to eliminate guardrail and, as a minimum, to be in accordance with Item 2 above. An evaluation of run-off-the-road accidents will assist in the assessment (see Chapter 50). See Section 54-4.0 for more information on roadside-safety criteria.

54-3.03(05) Right of Way

Where practical, additional right of way should be secured to permit cost-effective geometric and roadside-safety improvements.

54-3.03(06) Interchange

The project may include proposed work on an interchange. This work will only include selective improvements to the interchange geometrics. This may include lengthening acceleration or deceleration lanes, clearing the gore area, correcting the ramp superelevation, etc. The designer should consider the following.

1. Desirable. The criteria provided in Chapter 48 should be used to design each interchange element which will be improved as part of the freeway project.

2. Minimum. The criteria provided in the AASHTO A Policy on Geometric Design of Highways and Streets may be used as the minimum design where INDOT’s criteria exceed AASHTO’s. For example, Figures 54-3A and 54-3B may be used to determine deceleration distance for a freeway exit instead of INDOT’s standard 1000-ft length.

3. Acceleration or Deceleration Lane. Only a parallel ramp exit or entrance should be used; see Section 48-4.0. If converting a taper design to the preferred parallel design, the existing taper portion that is less than 12 ft wide should be removed and reconstructed to provide the full 12-ft width for the entire acceleration or deceleration length.
4. **Ramp Shoulder.** Under restrictive conditions, an existing right-hand-side shoulder width of 7.5 ft may be retained.

## 54-4.0 ROADSIDE SAFETY

The project should be evaluated for potential roadside-safety improvements within the project limits. The criteria described in Chapter 49 will apply to the evaluation. This includes roadside clear zone, barrier warrants as shown in Figure 49-4G(1), barrier design, and drainage features.

Not all 2:1 fill slopes or foreslopes will require the use of guardrail. The designer first should conduct a cost-effective analysis based on traffic volume, design speed, accident frequency, accident cost, accident severity, installation costs, and repair costs to determine if guardrail is necessary. Section 49-11.0 provides information on the AASHTO computer software program entitled ROADSIDE, which should be used for the cost-effectiveness analysis.

## 54-5.0 BRIDGES

### 54-5.01 General

Figure 54-2A provides the Department’s criteria for structural capacity and width for a new or reconstructed bridge, or for an existing bridge to remain in place. An existing bridge may remain in place if it meets, or is upgraded to meet, the structural and geometric requirements described in Figure 54-2A and Section 54-5.02. Upgrading a bridge to be in accordance with these criteria may be considered if an engineering analysis determines that the upgrading is appropriate. Some of the items that should be considered in the analysis include the following:

1. remaining service life;
2. sufficiency rating;
3. traffic volume;
4. clear-roadway width;
5. design speed; and
6. accident records.

If it is determined that a bridge should be replaced or undergo major reconstruction (e.g., replacing superstructure, widening superstructure or substructure), the design will be in accordance with the AASHTO LRFD criteria and load-carrying capacity (see Part VI).
54-5.02 Bridge To Remain In Place

An existing bridge should be evaluated for possible upgrading or replacement (see Section 54-5.01), if it is not in accordance with the following.

1. **Width.** The width should be evaluated against the criteria shown in Figure 54-2A.

2. **Structural Capacity.** The structural capacity should be evaluated against the criteria shown in Figure 54-2A.

3. **Vertical Clearance.** An existing structure should provide at least a 16-ft vertical clearance over the entire roadway including the usable shoulder widths for both left and right shoulders. If it is necessary to retain a vertical clearance of less than 16 ft, a design-exception request is required as described in Section 40-8.0. However, Section 54-3.02 provides a list of routes for which existing an overpass with a minimum 14-ft vertical clearance may be retained without a design exception.

4. **Bridge Railing.** Only an existing bridge railing that have been proven to be acceptable through crash testing may be retained. Each new bridge-railing installation should in accordance with Section 61-6.01. Consideration should be given to widening the bridge at the same time the railing is replaced to achieve the full approach-roadway width.

5. **Approach-Barrier Transition.** An approaching barrier transition should be in accordance with Chapter 49 and the INDOT Standard Drawings.

54-6.0 MEDIAN OPENING

On a fully access-controlled freeway, median crossing is denied to the public. However, an occasional median opening or emergency crossover is required to accommodate maintenance, snowplowing, or emergency service vehicles.

54-6.01 Guidelines

A median crossover should be placed away from a mainline conflict, such as an interchange. The number and location of median crossovers should be kept to a minimum.

1. **Spacing.** A median opening may be provided if it is in accordance with the spacing requirements as follows:
a. A median crossover may be provided approximately half way between two interchanges if the spacing between them is greater than 3 mi but less than 4 mi.

b. Multiple crossovers may be provided such that the distance between each crossover or interchange is not greater than 3 mi if the spacing between interchanges is greater than 4 mi.

2. **Jurisdiction.** A median crossover may be appropriate at a State line or a division line between districts or subdistricts.

3. **Urban.** A crossover should not be located in an urban area or an area with a narrow median.

4. **Interstate-Route Usage.** Section 54-6.04 provides a listing of the FHWA-approved crossover sites on the Interstate System.

5. **Rest Area or Weigh Station.** A crossover should be located at least 1500 ft from the end of the exit or entrance ramp taper for a rest area or weigh station.

**54-6.02 Implementation**

If warranted as discussed in Section 54-6.01, each new crossover on an existing facility should be in accordance with 54-6.03. The addition of a median crossover, either during construction or after the highway is in use, requires the approval of the Chief Engineer and concurrence from the FHWA.

**54-6.03 Design**

The INDOT *Standard Drawings* provide the criteria for the design of a freeway crossover. The designer should also consider the following.

1. **Interchange or Lane Drop.** A crossover should be at least 1500 ft from the terminus of an exit ramp, entrance ramp, or lane drop.

2. **Overhead Structure.** A crossover should be located at least 1500 ft from a structure crossing over the freeway.

3. **Sight Distance.** Because of unexpected nature of a U-turn maneuver, adequate sight distance should be available at a crossover. Decision sight distance should be provided in both
directions. This would favor, for example, placing the crossover in a sag vertical curve. The minimum stopping sight distance should be provided.

4. **Median Barrier.** A crossover should be avoided where a median barrier is present. If a crossover must be provided, the barrier should be flared as shown in Figure 54-6A, or terminated with an appropriate end treatment as discussed in Chapter 49. The width of the opening should not be greater than 35 ft.

6. **Horizontal Curve.** A crossover should not be located within a curve requiring superelevation.

7. **Pavement.** The crossover pavement will be constructed with an asphalt surface of sufficient strength to accommodate the largest expected vehicle (e.g., fully loaded dump truck, fire truck). See Chapter 52.

8. **Drainage.** A crossover should be located such that an additional drainage structure would not be required. The designer should review the median drainage patterns to ensure that the median crossover will not negatively disrupt the median drainage (e.g., cause ponding in the median). If a culvert is required under the crossover, consideration should be given to providing inlets or culvert end sections which are in accordance with Section 49-3.03 and the INDOT *Standard Drawings*.

### 54-6.04 Location of Interstate-Route Crossover

The FHWA-approved sites for median crossovers on the Interstate system are listed in Figure 54-6B(64) for I-64, Figure 54-6B(65) for I-65, Figure 54-6B(69) for I-69, Figure 54-6B(70) for I-70, Figure 54-6B(74) for I-74, Figure 54-6B(94) for I-94, or Figure 54-6B(100) for the three-digit-numbered routes. The sites are listed according to district, reference marker, and location description.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Rural</th>
<th>Urban</th>
</tr>
</thead>
<tbody>
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<td>Design Forecast Year</td>
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<td>20 Years (1)</td>
<td>20 Years (1)</td>
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<td>Desirable: B; Minimum: D</td>
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<td><strong>Cross Section Elements</strong></td>
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<td>Asphalt / Concrete</td>
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<td>2 Lanes: 4 ft Paved; 3 Lanes: 10 ft Paved</td>
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<td>Fill</td>
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<td></td>
<td>Sign Truss / Pedestrian Bridge</td>
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<td>New: 17.5 ft; Existing: 17 ft</td>
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* Controlling design criterion (see Section 40-8.0).

**GEOMETRIC DESIGN CRITERIA FOR FREEWAY**
*(3R or Partial 4R Project)*

**Figure 54-2A**
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Rural</th>
<th>Urban</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td></td>
<td>70 mph</td>
<td>55 mph</td>
</tr>
<tr>
<td></td>
<td></td>
<td>53 mph</td>
<td>60 mph</td>
</tr>
<tr>
<td></td>
<td></td>
<td>610 ft</td>
<td>70 mph</td>
</tr>
<tr>
<td>*Stopping Sight Distance</td>
<td>42-1.0</td>
<td>730 ft</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>*Minimum Radius</td>
<td>43-2.0</td>
<td>Existing (14)</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>*Superelevation Rate (15)</td>
<td>43-3.0</td>
<td>$e_{max} = 8%$</td>
<td>$e_{max} = 8%$</td>
</tr>
<tr>
<td>*Horizontal Sight Distance</td>
<td>43-4.0</td>
<td>See Section 43-4.0</td>
<td>See Section 43-4.0</td>
</tr>
<tr>
<td>*Vertical Curvature (K-value)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crest</td>
<td>44-3.0</td>
<td>Existing (14)</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>Sag</td>
<td></td>
<td>Existing (14)</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>*Maximum Grade</td>
<td>54-3.02</td>
<td>Existing (14)</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>Level</td>
<td></td>
<td>Existing (14)</td>
<td>Existing (14)</td>
</tr>
<tr>
<td>Rolling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>Desirable: 0.5%; Minimum: 0.0%</td>
<td>Desirable: 0.5%; Minimum: 0.0%</td>
</tr>
<tr>
<td>Width</td>
<td>48-5.02</td>
<td>16 ft</td>
<td>16 ft</td>
</tr>
<tr>
<td>Shoulder</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Width</td>
<td>48-5.02</td>
<td>Usable: 11 ft, Paved: Des: 8 ft; Min: 7.5 ft</td>
<td>Usable: 11 ft, Paved: Des: 8 ft; Min: 7.5 ft</td>
</tr>
<tr>
<td>Left Width</td>
<td></td>
<td>Usable: 7 ft, Paved: Des: 4 ft; Min: 2.5 ft</td>
<td>Usable: 7 ft, Paved: Des: 4 ft; Min: 2.5 ft</td>
</tr>
<tr>
<td>Surface Type (3)</td>
<td>Chp. 52</td>
<td>Asphalt / Concrete</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>Cross Slope</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traveled Way</td>
<td>48-5.02</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>Shoulder (17)</td>
<td></td>
<td>Right: 4%; Left: 2%</td>
<td>Right: 4%; Left: 2%</td>
</tr>
<tr>
<td>Surf. Type (16)</td>
<td>Chp. 52</td>
<td>Asphalt / Concrete</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>Superelevation</td>
<td>48-5.03</td>
<td>$e_{max} = 8%$</td>
<td>$e_{max} = 4%, 6%, \text{or } 8%$ (18)</td>
</tr>
<tr>
<td>Maximum Grade</td>
<td>48-5.04</td>
<td>3% - 5%</td>
<td>3% - 5%</td>
</tr>
<tr>
<td>Upgrade</td>
<td></td>
<td>4% - 6%</td>
<td>4% - 6%</td>
</tr>
<tr>
<td>Downgrade</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Controlling design criterion (see Section 40-8.0).

GEOMETRIC DESIGN CRITERIA FOR FREEWAY
(3R or Partial 4R Project)

Figure 54-2A (Continued)
(1) **Design Forecast Year.** A resurfaced pavement has a 10-year design life.

(2) **Design Speed.** The existing posted speed limit may be used in restricted urban conditions. The design speed should be 50 mph or higher on an Interstate highway.

(3) **Surface Type.** The pavement type selection will be determined by the Office of Pavement Engineering.

(4) **Shoulder Width, Right.** The following will apply.
   
a. The shoulder is paved to the face of guardrail. The desirable guardrail offset is 2 ft from the effective usable-shoulder width. See Section 49-5.0 for more information.

   b. If the number of trucks exceeds 250 DDHV, a 12-ft right shoulder should be considered. If the 12-ft shoulder is used, the usable-shoulder width will be 13 ft.

   c. Usable-shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(5) **Shoulder Width, Left.** The following will apply.

   a. The effective usable-shoulder width is equal to the paved-shoulder width. The desirable guardrail offset is 2 ft from the effective usable-shoulder width. See Section 49-5.0 for more information.

   b. Where there are 3 or more lanes in one direction, a 12-ft left shoulder should be provided if practical.

   c. Usable-shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point. Usable width is 1 ft wider than the paved-shoulder width.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(6A) **Cross Slope, Shoulder.** See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information.
(7) **Shoulders for Bridge to Remain in Place.** For such a bridge of length > 200 ft, the minimum shoulder width on the right and the left may be 3.5 ft. This requirement does not apply to a bridge deck replacement.

(8) **Clear Zone.** The clear zone will vary according to design speed, traffic volume, side slopes and horizontal curvature. See Section 49-2.0.

(9) **Side Slopes.** Retention of an existing side slope of 2:1 or flatter most often will be acceptable. However, an existing fill slope of steeper than 4:1 should be evaluated for flattening. Section 54-3.03 provides additional information for side-slope criteria for a project with freeway widening (i.e., lane or shoulder widening).

(10) **Structural Capacity (New or Reconstructed Bridge).** HS-25 loading with Alternate Military Loading should be applied for each project with notice to proceed with design beginning September 1, 2004, through December 31, 2005. Other loadings will apply to the Toll Road or an Extra Heavy Duty Highway. See Chapter Sixty for more information.

(11) **Width (New or Reconstructed Bridge).** See Sections 45-4.01 and 59-1.0 for more information on bridge width.

(12) **Vertical Clearance (Freeway Under).** The following will apply.
   a. Vertical clearance applies from usable edge to usable edge of shoulders.
   b. Table value includes an additional 0.5 ft allowance for future overlays.
   c. A 14-ft clearance may be used in an urban area where an alternative freeway facility with a 16-ft clearance is available; see Section 54-3.02.

(13) **Vertical Clearance (Freeway Over Railroad).** See Chapter Sixty-nine for additional information on railroad clearance under a highway.

(14) **Existing Conditions.** For these design elements, the existing conditions are satisfactory unless accident history dictates that a modification is necessary.

(15) **Superelevation Rate.** The designer should review Sections 43-2.0 and 43-3.0 to determine if improvements are necessary.

(16) **Shoulders (Surface Type).** The pavement-type selection will be determined by the Office of Pavement Engineering. For a ramp with curve radii less than or equal to 350 ft, the shoulders will have the same pavement design as the travelway.

(17) **Cross Slope (Shoulders).** For a ramp with curve radii less than or equal to 350 ft, the shoulder cross slope will be the same as the travelway.

(18) **Superelevation.** The maximum superelevation rate will depend on site conditions. The highest rate practical should be used, especially for a descending ramp.
### Highway Design Speed (mph) Average Running Speed, \( V_a \) (mph)

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Average Running Speed, ( V_a ) (mph)</th>
<th>( L = ) Deceleration Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>For Design Speed of First Governing Geometric Control (mph)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stop 15 20 25 30 40 45 50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>For Average Running Speed, ( V'_a ) (mph)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0 10 17 22 26 32 39 43</td>
</tr>
<tr>
<td>30</td>
<td>29</td>
<td>250 230 200 150 -- -- -- --</td>
</tr>
<tr>
<td>40</td>
<td>34</td>
<td>320 300 270 220 180 -- -- -- --</td>
</tr>
<tr>
<td>45</td>
<td>39</td>
<td>360 350 310 280 230 180 -- -- -- --</td>
</tr>
<tr>
<td>50</td>
<td>43</td>
<td>430 410 380 360 300 270 180 -- -- -- --</td>
</tr>
<tr>
<td>55</td>
<td>48</td>
<td>480 460 450 400 360 330 250 200</td>
</tr>
<tr>
<td>60</td>
<td>53</td>
<td>560 550 510 480 450 400 330 280</td>
</tr>
<tr>
<td>70</td>
<td>57</td>
<td>590 590 560 530 500 460 400 350</td>
</tr>
</tbody>
</table>

**Notes:**

1. Value is for a grade of 3% or less. See Figure 54-3B for steeper upgrades or downgrades.

2. The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the highway mainline speed to the speed of the first governing geometric control on the exit ramp. The basic assumptions within the AASHTO deceleration model are as follows.

   a. The vehicle is initially traveling at the average running speed of the highway mainline.

   b. The vehicle decelerates in gear for 3 s of travel time.

   c. The motorist brakes the vehicle at a comfortable rate until it reaches the average running speed of the first governing geometric control.

*The AASHTO deceleration model is discussed in detail in A Policy on Geometric Design of Rural Highways, 1965, pp. 348-351.*

**LENGTH FOR DECELERATION**

Figure 54-3A
Direction of Grade & Ratio of Deceleration Length on Grade to Length on Level

<table>
<thead>
<tr>
<th>Direction of Grade</th>
<th>&lt; 3%</th>
<th>3% ≤ Ratio &lt; 4%</th>
<th>4% ≤ Ratio &lt; 6%</th>
<th>≥ 6%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upgrade*</td>
<td>1.00</td>
<td>0.90</td>
<td>0.80</td>
<td>0.70</td>
</tr>
<tr>
<td>Downgrade</td>
<td>1.00</td>
<td>1.20</td>
<td>1.35</td>
<td>1.50</td>
</tr>
</tbody>
</table>

* Upgrade adjustment is only used in a restricted location.

Notes:
1. Table applies to each highway design speed.
2. The grade in the table is the average grade over the distance used for measuring the length of deceleration.

Example

Given: Highway Design Speed 70 mph
First Exit Curve Design Speed 45 mph
Average Grade 5% downgrade

Problem: Determine length of deceleration.

Solution: Figure 54-3A yields a minimum deceleration length of 400 ft on the level. According to Figure 54-3B, this should be increased by 1.35.

Therefore:

\[ L = (400 \text{ ft}) (1.35) \]
\[ L = 540 \text{ ft} \]
BARRIER TREATMENT AT A MEDIAN CROSSOVER

Figure 54-6A
<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>I-64 Location Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vincennes District</td>
</tr>
<tr>
<td>0.8</td>
<td>0.8 mi. E. of Indiana-Illinois State Line</td>
</tr>
<tr>
<td>3.7</td>
<td>0.7 mi. W. of Griffin Road</td>
</tr>
<tr>
<td>5.1</td>
<td>0.7 mi. E. of Griffin Road</td>
</tr>
<tr>
<td>6.9*</td>
<td>2.5 mi. E. of Griffin Road (0.4 mi. W. of Black River Rest Area)</td>
</tr>
<tr>
<td>8.9</td>
<td>4.5 mi. E. of Griffin Road (1.2 mi. E. of Black River Rest Area)</td>
</tr>
<tr>
<td>10.9</td>
<td>1.0 mi. W. of SR 165</td>
</tr>
<tr>
<td>12.4*</td>
<td>0.5 mi. E. of SR 165</td>
</tr>
<tr>
<td>16.9</td>
<td>0.8 mi. W. of SR 65</td>
</tr>
<tr>
<td>18.2</td>
<td>0.5 mi. E. of SR 65</td>
</tr>
<tr>
<td>19.8</td>
<td>2.1 mi. E. of SR 65</td>
</tr>
<tr>
<td>23.3</td>
<td>1.7 mi. W. of US 41</td>
</tr>
<tr>
<td>25.8*</td>
<td>0.8 mi. E. of US 41</td>
</tr>
<tr>
<td>28.8</td>
<td>0.6 mi. W. of SR 57 &amp; I-164</td>
</tr>
<tr>
<td>30.2*</td>
<td>0.8 mi. E. of SR 57 &amp; I-164</td>
</tr>
<tr>
<td>33.3</td>
<td>3.9 mi. E. of SR 57 &amp; I-164</td>
</tr>
<tr>
<td>36.8</td>
<td>2.5 mi. W. of SR 61</td>
</tr>
<tr>
<td>38.7</td>
<td>0.6 mi. W. of SR 61</td>
</tr>
<tr>
<td>40.2</td>
<td>0.9 mi. E. of SR 61</td>
</tr>
<tr>
<td>43.6</td>
<td>4.3 mi. E. of SR 61</td>
</tr>
<tr>
<td>46.9*</td>
<td>7.6 mi. E. of SR 61</td>
</tr>
<tr>
<td>50.2</td>
<td>3.4 mi. W. of SR 161</td>
</tr>
<tr>
<td>53.1*</td>
<td>0.5 mi. W. of SR 161</td>
</tr>
<tr>
<td>54.3*</td>
<td>0.6 mi. E. of SR 161</td>
</tr>
<tr>
<td>55.9</td>
<td>0.9 mi. W. of US 231</td>
</tr>
<tr>
<td>57.7</td>
<td>1.0 mi. E. of US 231 (0.3 mi. W. of Dale Rest Area)</td>
</tr>
<tr>
<td>59.9</td>
<td>3.1 mi. W. of SR 162 (1.4 mi. E. of Dale Rest Area)</td>
</tr>
<tr>
<td>62.3</td>
<td>0.7 mi. W. of SR 162</td>
</tr>
<tr>
<td>63.7</td>
<td>0.7 mi. E. of SR 162</td>
</tr>
<tr>
<td>68.6*</td>
<td>5.6 mi. W. of SR 162</td>
</tr>
<tr>
<td>71.7</td>
<td>0.7 mi. W. of SR 145</td>
</tr>
<tr>
<td>73.0*</td>
<td>0.6 mi. E. of SR 145</td>
</tr>
<tr>
<td>75.5*</td>
<td>3.1 mi. E. of SR 145</td>
</tr>
<tr>
<td>78.0</td>
<td>0.6 mi. W. of West Junction SR 37</td>
</tr>
<tr>
<td>79.2</td>
<td>1.8 mi. W. of West Junction SR 37</td>
</tr>
</tbody>
</table>

* New crossover location

**INTERSTATE-ROUTE CROSSEORS, I-64**

**Figure 54-6B(64)**
Ref. Mkr. | I-64 Location Description
--- | ---
**Vincennes District (cont’d.)**
84.7 | 1.1 mi. of East Junction SR 37
86.5 | 0.7 mi. E. of East Junction SR 37
90.0 | 2.1 mi. W. of SR 66

**Seymour District**

92.5 | 04. mi. E. of SR 66
94.9 | 2.8 mi. E. of SR 66
98.5 | 6.4 mi. E. of SR 66
101.9 | 9.8 mi. E. of SR 66
104.8* | 0.7 mi. E. of SR 135
105.9 | 0.4 mi. E. of SR 135
109.2 | 3.7 mi. E. of SR 135
111.9 | 0.7 mi. W. of Lanesville Road
113.4* | 0.8 mi. E. of Lanesville Road (1.1 mi. W. of Lanesville Rest Area)
117.1* | 0.8 mi. E. of SR 62 & SR 64 (1.9 mi. E. of Lanesville Rest Area)
118.7* | 0.8 mi. E. of SR 62 & SR 64 (0.9 mi. W. of US 150)
119.8* | 0.2 mi. S. of US 150
121.3 | 0.5 mi. W. of I-265
122.6 | 0.8 mi. E. of I-265 (0.8 mi. W. of Spring Street)

* New crossover location

**INTERSTATE-ROUTE CROSSEOVERS, I-64**

**Figure 54-6B(64)** (Continued)
**Ref. Mkr.** | **I-65 Location Description**
--- | ---

**Seymour District**

<table>
<thead>
<tr>
<th>Mileage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.7</td>
<td>0.6 mi. N. of SR 311</td>
</tr>
<tr>
<td>12.5</td>
<td>3.4 mi. N. of SR 311</td>
</tr>
<tr>
<td>15.1*</td>
<td>0.6 mi. S. of Memphis Road</td>
</tr>
<tr>
<td>16.4*</td>
<td>0.7 mi. N. of Memphis Road</td>
</tr>
<tr>
<td>18.5</td>
<td>0.7 mi. S. of SR 160</td>
</tr>
<tr>
<td>19.9</td>
<td>0.7 mi. N. of SR 160</td>
</tr>
<tr>
<td>21.6*</td>
<td>2.4 mi. N. of SR 160 (0.5 mi. S. of Henryville NB Rest Area)</td>
</tr>
<tr>
<td>23.0*</td>
<td>3.8 mi. N. of SR 160 (0.5 mi. N. of Henryville SB Rest Area)</td>
</tr>
<tr>
<td>26.2</td>
<td>7.0 mi. N. of SR 160</td>
</tr>
<tr>
<td>28.7</td>
<td>0.6 mi. S. of SR 56</td>
</tr>
<tr>
<td>29.9</td>
<td>0.6 mi. N. of SR 56</td>
</tr>
<tr>
<td>32.8*</td>
<td>0.7 mi. S. of SR 256</td>
</tr>
<tr>
<td>34.2*</td>
<td>0.7 mi. N. of SR 256</td>
</tr>
<tr>
<td>35.7*</td>
<td>0.7 mi. S. of US 31</td>
</tr>
<tr>
<td>37.3</td>
<td>0.8 mi. N. of US 31</td>
</tr>
<tr>
<td>40.3*</td>
<td>0.8 mi. S. of SR 250</td>
</tr>
<tr>
<td>41.8*</td>
<td>0.7 mi. N. of SR 250</td>
</tr>
<tr>
<td>45.5</td>
<td>0.2 mi. N. of US 31</td>
</tr>
<tr>
<td>48.7</td>
<td>0.8 mi. S. of US 50</td>
</tr>
<tr>
<td>50.6*</td>
<td>1.1 mi. N. of US 150</td>
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<tr>
<td>54.5</td>
<td>0.7 mi. S. of SR 11</td>
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<tr>
<td>56.1</td>
<td>0.9 mi. N. of SR 11</td>
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<tr>
<td>59.5*</td>
<td>4.3 mi. N. of SR 11</td>
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<tr>
<td>63.1</td>
<td>0.6 mi. N. of SR 58</td>
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<td>64.3</td>
<td>0.6 mi. S. of SR 58</td>
</tr>
<tr>
<td>67.6</td>
<td>0.6 mi. N. of SR 46</td>
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<td>68.9</td>
<td>0.6 mi. N. of SR 46</td>
</tr>
<tr>
<td>71.6</td>
<td>3.3 mi. N. of SR 46 (0.5 mi. S. of Taylorsville NB Rest Area)</td>
</tr>
<tr>
<td>74.5*</td>
<td>1.2 mi. S. of US 31 (0.7 mi. N. of Taylorsville SB Rest Area)</td>
</tr>
<tr>
<td>76.7</td>
<td>1.0 mi. N. of US 31</td>
</tr>
<tr>
<td>79.3*</td>
<td>0.8 mi. S. of SR 252</td>
</tr>
<tr>
<td>80.8</td>
<td>0.7 mi. N. of SR 252</td>
</tr>
<tr>
<td>84.1</td>
<td>4.0 mi. N. of SR 252</td>
</tr>
<tr>
<td>88.9</td>
<td>0.7 mi. S. of SR 44</td>
</tr>
</tbody>
</table>

* New crossover location

**INTERSTATE-ROUTE CROSSEOVERS, I-65**

*Figure 54-6B(65)*
Ref. Mkr. | I-65 Location Description
--- | ---

**Seymour District (cont’d.)**

<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>90.2</td>
<td>0.6 mi. N. of SR 44</td>
</tr>
<tr>
<td>93.7</td>
<td>0.8 mi. S. of Whiteland Road</td>
</tr>
<tr>
<td>95.3*</td>
<td>0.8 mi. N. of Whiteland Road</td>
</tr>
<tr>
<td>97.9</td>
<td>1.4 mi. S. of Greenwood Road</td>
</tr>
<tr>
<td>100.6</td>
<td>1.3 mi. N. of Greenwood Road</td>
</tr>
</tbody>
</table>

**Greenfield District**

<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>102.2</td>
<td>0.9 mi. S. of Southport Road</td>
</tr>
<tr>
<td>104.9</td>
<td>1.1 mi. S. of I-465 South Leg</td>
</tr>
<tr>
<td>119.7*</td>
<td>1.2 mi. S. of Lafayette Road</td>
</tr>
<tr>
<td>121.7*</td>
<td>0.8 mi. N. of Lafayette Road</td>
</tr>
<tr>
<td>123.9*</td>
<td>0.5 mi. S. of 71st Street</td>
</tr>
<tr>
<td>124.9*</td>
<td>0.5 mi. N. of 71st Street</td>
</tr>
<tr>
<td>127.8*</td>
<td>3.4 mi. N. of 71st Street</td>
</tr>
<tr>
<td>129.7</td>
<td>0.4 mi. S. of SR 334</td>
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**Crawfordsville District**

<table>
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<th>Description</th>
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</thead>
<tbody>
<tr>
<td>130.6*</td>
<td>0.5 mi. N. of SR 334</td>
</tr>
<tr>
<td>132.8</td>
<td>0.5 mi. S. of SR 267</td>
</tr>
<tr>
<td>133.8</td>
<td>0.5 mi. N. of SR 267</td>
</tr>
<tr>
<td>136.8*</td>
<td>0.9 mi. S. of SR 100E</td>
</tr>
<tr>
<td>138.3*</td>
<td>0.5 mi. S. of SR 39</td>
</tr>
<tr>
<td>141.1*</td>
<td>1.0 mi. N. of SR 32</td>
</tr>
<tr>
<td>142.4*</td>
<td>0.4 mi. N. of US 52</td>
</tr>
<tr>
<td>145.1</td>
<td>0.7 mi. of S. of SR 47</td>
</tr>
<tr>
<td>146.6</td>
<td>0.8 mi. N. of SR 47 (1.8 mi. S. of Lebanon NB Rest Area)</td>
</tr>
<tr>
<td>151.0</td>
<td>5.2 mi. N. of SR 47 (1.1 mi. N. of Lebanon SB Rest Area)</td>
</tr>
<tr>
<td>153.7*</td>
<td>4.2 mi. S. of SR 28</td>
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<tr>
<td>157.2</td>
<td>0.7 mi. S. of SR 28</td>
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</table>

* New crossover location

**INTERSTATE-ROUTE Crossovers, I-65**

*Figure 54-B(65) (Continued)*
Ref. Mkr.  I-65 Location Description

158.8*  0.9 mi. N. of SR 28
        Crawfordsville District (cont’d.)

162.0  4.1 mi. N. of SR 28
165.5  3.0 mi. S. of SR 38
167.9  0.6 mi. S. of SR 38
169.3  0.8 mi. N. of SR 38
171.6  0.5 mi. S. of SR 26
172.9  0.8 mi. N. of SR 26
174.2  0.9 mi. S. of SR 25
176.6*  1.5 mi. N. of SR 25
178.9  0.6 mi. N. of US 231
181.5  3.2 mi. N. of US 231
184.9  3.1 mi. S. of SR 18
187.0  1.0 mi. S. of SR 18
188.7  0.7 mi. N. of SR 18
192.7  0.6 mi. S. of US 231
194.3  1.0 mi. N. of US 231 (1.4 mi. S. of Wolcott SB Rest Area)
197.1*  4.3 mi. S. of SR 24 (0.9 mi. N. of Wolcott SB Rest Area)
200.4  1.0 mi. of S. of SR 24

LaPorte District

202.1*  0.6 mi. N. of US 24
203.9*  0.8 mi. S. of US 231
205.6*  0.6 mi. N. of US 231
207.7  2.7 mi. N. of US 231
209.6  0.3 mi. N. of SR 16
211.7  2.1 mi. N. of SR 16
214.0  0.7 mi. S. of SR 114
215.4  0.7 mi. N. of SR 114
217.8  3.1 mi. N. of SR 114
219.9  0.6 mi. S. of SR 14
223.2  2.8 mi. N. of SR 14
225.1*  4.6 mi. N. of SR 14

*  New crossover location

INTERSTATE-ROUTE CROSSOVERS, I-65

Figure 54-B(65) (Continued)
<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>I-65 Location Description</th>
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<tbody>
<tr>
<td>228.9</td>
<td>0.7 mi. S. of SR 10</td>
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<td>LaPorte District (cont’d.)</td>
</tr>
<tr>
<td>230.2</td>
<td>0.5 mi. N. of SR 10 (0.7 mi. S. of Kankakee NB Rest Area)</td>
</tr>
<tr>
<td>232.9</td>
<td>3.3 mi. N. of SR 10 (1.6 mi. N. of Kankakee NB Rest Area)</td>
</tr>
<tr>
<td>235.8</td>
<td>4.2 mi. S. of SR 2</td>
</tr>
<tr>
<td>239.3</td>
<td>0.6 mi. S. of SR 2</td>
</tr>
<tr>
<td>241.8*</td>
<td>1.6 mi. N. of SR 2 (0.6 mi. N. of NB Weigh Station)</td>
</tr>
<tr>
<td>244.1</td>
<td>3.3 mi. S. of US 231</td>
</tr>
<tr>
<td>246.8</td>
<td>0.7 mi. S. of US 231</td>
</tr>
<tr>
<td>247.8*</td>
<td>0.4 mi. N. of US 231</td>
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<tr>
<td>250.7</td>
<td>3.3 mi. N. of US 231</td>
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<tr>
<td>252.1</td>
<td>0.7 mi. S. of US 30</td>
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<td>260.8</td>
<td>1.1 mi. N. of I-80 &amp; I-94</td>
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</table>

* New crossover location

**INTERSTATE-ROUTE CROSOVERS, I-65**

Figure 54-B(65) (Continued)
<table>
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<td>6.3</td>
<td>1.3 mi. N. of 116th Street</td>
</tr>
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<td>8.1</td>
<td>2.0 mi. S. of SR 238</td>
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<td>10.9*</td>
<td>0.8 mi. N. of SE 238</td>
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<tr>
<td>13.7</td>
<td>0.8 mi. S. of SR 13</td>
</tr>
<tr>
<td>15.2</td>
<td>0.7 mi. N. of SR 13</td>
</tr>
<tr>
<td>18.2</td>
<td>0.5 mi. S. of SR 38</td>
</tr>
<tr>
<td>19.3*</td>
<td>0.6 mi. N. of SR 38</td>
</tr>
<tr>
<td>21.4</td>
<td>1.0 mi. S. of SR 67 &amp; SR 9</td>
</tr>
<tr>
<td>23.0</td>
<td>0.6 mi. N. of SR 67 &amp; SR 9</td>
</tr>
<tr>
<td>25.2</td>
<td>1.0 mi. S. of N. Jct. SR 9</td>
</tr>
<tr>
<td>27.0*</td>
<td>0.8 mi. N. of N. Jct. SR 9</td>
</tr>
<tr>
<td>29.3</td>
<td>3.0 mi. N. of N. Jct. SR 9</td>
</tr>
<tr>
<td>32.5</td>
<td>1.0 mi. S. of SR 32 &amp; SR 67</td>
</tr>
<tr>
<td>34.3</td>
<td>0.9 mi. N. of SR 32 &amp; SR 67</td>
</tr>
<tr>
<td>37.3</td>
<td>3.9 mi. N. of SR 32 &amp; SR 67</td>
</tr>
<tr>
<td>40.3</td>
<td>0.4 mi. S. of SR 332</td>
</tr>
<tr>
<td>42.1*</td>
<td>1.4 mi. N. of SR 332</td>
</tr>
<tr>
<td>44.1</td>
<td>0.6 mi. S. of US 35 &amp; SR 28</td>
</tr>
<tr>
<td>45.3</td>
<td>0.6 mi. N. of US 35 &amp; SR 28</td>
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<tr>
<td>49.4</td>
<td>4.7 mi. N. of US 35 &amp; SR 28</td>
</tr>
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<td>51.3*</td>
<td>3.8 mi. S. of SR 26 (0.5 mi. N. of Pipe Creek SB Rest Area)</td>
</tr>
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<td>54.3*</td>
<td>0.8 mi. S. of SR 26</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>55.5*</td>
<td>0.5 mi. N. of SR 26</td>
</tr>
<tr>
<td>58.0</td>
<td>1.0 mi. S. of SR 22</td>
</tr>
<tr>
<td>59.7*</td>
<td>0.7 mi. N. of SR 22</td>
</tr>
<tr>
<td>62.9</td>
<td>1.2 mi. S. of SR 18</td>
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<tr>
<td>64.8</td>
<td>0.7 mi. N. of SR 18</td>
</tr>
<tr>
<td>67.2</td>
<td>3.1 mi. N. of SR 18</td>
</tr>
<tr>
<td>69.7</td>
<td>3.1 mi. S. of SR 218</td>
</tr>
<tr>
<td>72.3</td>
<td>0.5 mi. S. of SR 218</td>
</tr>
</tbody>
</table>

* New crossover location
Ref. Mkr. | I-69 Location Description

Fort Wayne District (cont’d.)

73.4 | 0.6 mi. N. of SR 218
76.9* | 0.6 mi. S. of SR 5
78.5 | 1.0 mi. N. of SR 5 (1.2 mi. S. of Weigh Station)
80.5* | 3.0 mi. N. of SR 5 (0.4 mi. N. of Weigh Station)
83.5 | 2.9 mi. S. of US 224
85.8* | 0.6 mi. S. of US 224
87.6 | 1.2 mi. N. of US 224
88.7* | 2.3 mi. N. of US 224 (0.4 mi. S. of Flat Creek NB Rest Area)
90.3* | 3.9 mi. N. of US 224 (0.5 mi. N. of Flat Creek NB Rest Area)
92.2* | 4.3 mi. S. of SR 469 (0.4 mi. S. of Flat Creek SB Rest Area)
93.4 | 3.1 mi. S. of SR 469 (0.4 mi. N. of Flat Creek SB Rest Area)
95.5 | 1.0 mi. S. of SR 469
98.0 | 1.5 mi. N. of SR 469
100.5* | 1.5 mi. N. of Lower Huntington Road
103.7* | 1.7 mi. N. of US 24
106.9 | 1.6 mi. N. of SR 14
108.6 | 0.7 mi. S. of US 30 & US 33
113.1* | 0.9 mi. N. of Coldwater Road
116.4* | 0.6 mi. N. of SR 1
119.7* | 3.9 mi. N. of SR 1
123.4* | 2.8 mi. S. of CR 11A (0.5 mi. S. of Auburn NB Rest Area)
125.2* | 1.0 mi. S. of SR 11A (1.0 mi. N. of Auburn SB Rest Area)
128.6 | 0.4 mi. S. of SR 8
129.6* | 0.6 mi. N. of SR 8
132.9 | 1.4 mi. S. of US 6
134.8 | 0.5 mi. N. of US 6
137.0 | 2.7 mi. N. of US 6
140.0 | 0.4 mi. S. of SR 4
141.0 | 0.6 mi. N. of SR 4
142.8 | 2.4 mi. N. of SR 4 (1.0 mi. S. of Pigeon Creek SB Rest Area)
144.7* | 4.3 mi. N. of SR 4 (0.4 mi. N. of Pigeon Creek SB Rest Area)
147.4* | 0.5 mi. S. of US 20
149.0* | 1.0 mi. N. of US 20

* New crossover location

INTERSTATE-ROUTE CROSSES, I-69

Figure 54-6B(69) (Continued)
Ref. Mkr.  | I-69 Location Description
---|---
151.1  | 0.7 mi. N. of CR 200W
       | Fort Wayne District (cont’d.)
153.1* | 0.9 mi. S. of SR 127
155.1  | 1.1 mi. N. of SR 127
156.4* | 0.5 mi. S. of Jamestown Exit
157.6* | 0.7 mi. N. of Jamestown Exit (0.2 mi. S. of Indiana-Michigan State Line)

* New crossover location

INTERSTATE-ROUTE CROSSOVERS, I-69

Figure 54-6B(69) (Continued)
<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>I-70 Location Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>Indiana-Illinois State Line (1.4 mi. W. of Terre Haute Rest Area)</td>
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<tr>
<td>2.8</td>
<td>0.5 mi. W. of Darwin Road (1.3 mi. E. of Terre Haute Rest Area)</td>
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<tr>
<td>4.1</td>
<td>0.7 mi. E. of Darwin Road</td>
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<tr>
<td>5.8*</td>
<td>1.1 mi. W. of US 41</td>
</tr>
<tr>
<td>7.6</td>
<td>0.7 mi. E. of US 41</td>
</tr>
<tr>
<td>9.2</td>
<td>2.0 mi. W. of SR 46</td>
</tr>
<tr>
<td>10.5*</td>
<td>0.7 mi. W. of SR 46</td>
</tr>
<tr>
<td>12.6*</td>
<td>1.4 mi. E. of SR 46</td>
</tr>
<tr>
<td>16.2</td>
<td>5.0 mi. E. of SR 46</td>
</tr>
<tr>
<td>19.2*</td>
<td>3.4 mi. W. of SR 59</td>
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<tr>
<td>21.9*</td>
<td>0.7 mi. W. of SR 59</td>
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<tr>
<td>23.3</td>
<td>0.7 mi. E. of SR 59</td>
</tr>
<tr>
<td>27.1</td>
<td>4.5 mi. E. of SR 59</td>
</tr>
<tr>
<td>29.6</td>
<td>7.5 mi. W. of SR 243</td>
</tr>
<tr>
<td>32.8</td>
<td>4.4 mi. W. of SR 243</td>
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<tr>
<td>35.1</td>
<td>2.1 mi. W. of SR 243</td>
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<td>38.0</td>
<td>0.8 mi. E. of SR 243</td>
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<td>40.3*</td>
<td>0.7 mi. W. of US 231</td>
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<tr>
<td>42.0</td>
<td>0.9 mi. E. of US 231</td>
</tr>
<tr>
<td>45.2*</td>
<td>4.1 mi. E. of US 231</td>
</tr>
<tr>
<td>47.8</td>
<td>6.7 mi. E. of US 231</td>
</tr>
<tr>
<td>50.2</td>
<td>0.5 mi. W. of CR 1100W Little Point Road</td>
</tr>
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<td>51.5</td>
<td>0.7 mi. E. of CR 1100W Little Point Road</td>
</tr>
<tr>
<td>55.8</td>
<td>3.6 mi. W. of SR 39</td>
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<tr>
<td>58.8</td>
<td>0.6 mi. W. of SR 39</td>
</tr>
<tr>
<td>60.2*</td>
<td>0.8 mi. E. of SR 39</td>
</tr>
<tr>
<td>63.5</td>
<td>2.9 mi. W. of SR 267 (1.0 mi. W. of Plainfield Rest Area)</td>
</tr>
<tr>
<td>65.7</td>
<td>0.7 mi. W. of SR 267 (1.1 mi. E. of Plainfield Rest Area)</td>
</tr>
<tr>
<td>67.1</td>
<td>0.7 mi. E. of SR 267</td>
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<tr>
<td>69.2</td>
<td>2.8 mi. E. of SR 267</td>
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<tr>
<td>71.0</td>
<td>2.0 mi. W. of the W. leg I-465</td>
</tr>
</tbody>
</table>

* New crossover location

**INTERSTATE-ROUTE CROSSEORS, I-70**

*Figure 54-6B(70)*
Ref. Mkr. | I-70 Location Description
---|---

Greenfield District

90.2 | 0.6 mi. W. of Post Road
91.3* | 0.5 mi. E. of Post Road
93.9 | 3.1 mi. E. of Post Road
95.4 | 0.5 mi. W. of Mt. Comfort Road
96.8 | 0.9 mi. E. of Mt. Comfort Road
100.3* | 4.4 mi. E. of Mt. Comfort Road
102.9 | 0.8 mi. W. of SR 9
104.4 | 0.7 mi. E. of SR 9
106.4* | 2.7 mi. E. of SR 9 (0.4 mi. W. of Greenfield EB Rest Area)
107.6* | 3.9 mi. E. of SR 9 (0.4 mi. E. of Greenfield EB Rest Area)
110.7 | 7.0 mi. E. of SR 9
113.2* | 2.1 mi. W. of SR 109 (0.4 mi. W. of Greenfield WB Rest Area)
114.7 | 0.6 mi. W. of SR 109 (0.6 mi. E. of Greenfield WB Rest Area)
116.4 | 1.1 mi. E. of SR 109
119.8 | 4.5 mi. E. of SR 109
122.3 | 0.7 mi. W. of SR 3
123.5* | 0.5 mi. E. of SR 3
125.9 | 2.9 mi. E. of SR 3
128.9 | 2.1 mi. W. of Wilber Wright Road
131.6 | 0.6 mi. E. of Wilber Wright Road
133.8 | 2.8 mi. E. of Wilber Wright Road
136.7 | 0.7 mi. W. of SR 1
138.2* | 0.8 mi. E. of SR 1
140.3* | 2.9 mi. E. of SR 1
143.4* | 2.1 mi. W. of Centerville Road (0.4 mi. W. of Centerville Rest Area)
144.7 | 0.6 mi. W. of Centerville Road (0.7 mi. E. of Centerville Rest Area)
146.3* | 1.2 mi. E. of Centerville Road (1.4 mi. W. of Weigh Station)
153.5* | 0.8 mi. E. of SR 227
155.5 | 0.6 mi. W. of US 40 (0.7 mi. W. of Indiana-Ohio State Line)

* New crossover location

INTERSTATE-ROUTE CROSSES, I-70

Figure 54-6B(70) (Continued)
<table>
<thead>
<tr>
<th>Ref. Mkr.</th>
<th>Location Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>Indiana-Illinois State Line</td>
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<tr>
<td>0.6*</td>
<td>0.6 mi. E. of Indiana-Illinois State Line (0.6 mi. W. of Spring Creek Rest Area)</td>
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<tr>
<td>3.2</td>
<td>1.1 mi. W. of SR 63</td>
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<tr>
<td>4.8</td>
<td>0.5 mi. E. of SR 63</td>
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<td>7.3*</td>
<td>0.7 mi. W. of Covington/Strington Road</td>
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<td>8.7*</td>
<td>0.7 mi. E. of Covington/Strington Road</td>
</tr>
<tr>
<td>11.8*</td>
<td>3.7 mi. W. of US 41</td>
</tr>
<tr>
<td>14.4</td>
<td>1.1 mi. W. of US 41</td>
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<tr>
<td>16.2</td>
<td>0.7 mi. E. of US 41</td>
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<tr>
<td>17.3*</td>
<td>1.8 mi. E. of US 41 (1.0 mi. W. of Veedersburg Weigh Station)</td>
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<tr>
<td>19.5*</td>
<td>4.0 mi. E. of US 41 (0.8 mi. E. of Veedersburg Weigh Station)</td>
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<td>21.6*</td>
<td>6.1 mi. E. of US 41 (0.7 mi. W. of Waynetown Rest Area)</td>
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<td>25.5</td>
<td>0.6 mi. E. of SR 25</td>
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<tr>
<td>29.0</td>
<td>4.8 mi. W. of US 231</td>
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<tr>
<td>32.4</td>
<td>1.4 mi. W. of US 231</td>
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<td>35.2</td>
<td>1.4 mi. E. of US 231</td>
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<tr>
<td>38.6*</td>
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<td>0.7 mi. E. of SR 32</td>
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<td>4.7 mi. E. of SR 32</td>
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<td>47.3</td>
<td>4.7 mi. W. of SR 75</td>
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<td>51.4</td>
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<td>52.5</td>
<td>0.5 mi. E. of SR 75</td>
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<td>55.5*</td>
<td>3.5 mi. E. of SR 75 (0.8 mi. W. of Lizton Rest Area)</td>
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<td>57.1</td>
<td>0.5 mi. W. of SR 39 (0.4 mi. E. of Lizton Rest Area)</td>
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<td>58.0</td>
<td>0.4 mi. E. of SR 39</td>
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<tr>
<td>60.6*</td>
<td>0.6 mi. W. of Pittsboro Road</td>
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<td>61.8*</td>
<td>0.6 mi. E. of Pittsboro Road</td>
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<td>65.0</td>
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<td>66.5*</td>
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<tr>
<td>69.2</td>
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<tr>
<td>72.8</td>
<td>0.5 mi. W. of I-465</td>
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</tbody>
</table>

* New crossover location

**INTERSTATE-ROUTE CROSSTRAVES, I-74**

**Figure 54-6B(74)**
Ref. Mkr. I-74 Location Description

Greenfield District

94.5* 1.4 mi. W. of Post Road
97.1* 1.9 mi. W. of Acton Road
100.5 0.6 mi. W. of Pleasant View Road
101.8* 0.7 mi. E. of Pleasant View Road
103.6 0.6 mi. E. of London Road
106.3 3.3 mi. E. of London Road
108.8 0.6 mi. W. of Fairland Road
109.9 0.5 mi. E. of Fairland Road
112.1 0.7 mi. W. of SR 9
113.4 0.6 mi. E. of SR 9
117.1* 1.3 mi. W. of SR 244
119.1* 0.7 mi. E. of SR 244
122.6 0.5 mi. W. of St. Paul-Middletown Exit

Seymour District

123.9 0.7 mi. E. of St. Paul-Middletown Exit
126.8 3.6 mi. E. of St. Paul-Middletown Exit
129.2 6.0 mi. E. of St. Paul-Middletown Exit
131.5 0.5 mi. W. of US 421 WB Ramp
133.2 0.8 mi. W. of SR 3
134.7 0.6 mi. E. of SR 3
138.5* 4.4 mi. E. of SR 3
142.3* 0.7 mi. W. of New Point Road
143.7* 0.7 mi. E. of New Point Road
146.1* 3.1 mi. E. of New Point Road
148.4* 0.6 mi. W. of SR 229
149.6 0.6 mi. E. of SR 229
150.9* 1.9 mi. E. of SR 229 (0.5 mi. W. of Batesville EB Rest Area)
152.7* 3.7 mi. E. of SR 229 (0.6 mi. E. of Batesville WB Rest Area)
155.0 0.7 mi. W. of SR 101
156.5 0.8 mi. E. of SR 101

* New crossover location

INTERSTATE-ROUTE CROSSES, I-74

Figure 54-6B(74) (Continued)
Ref. Mkr. | I-74 Location Description
---|---
159.4* | 3.7 mi. E. of SR 101  
  | Seymour District (cont’d.)
162.3* | 1.3 mi. W. of SR 1  
164.3 | 0.7 mi. E. of SR 1  
166.7* | 3.1 mi. E. of SR 1  
168.1* | 0.9 mi. W. of US 52  
169.9* | 0.9 mi. E. of US 52 (0.6 mi. W. of Weigh Station)  
171.2 | 2.2 mi. E. of US 52  
  | (0.3 mi. E. of Weigh Station) (0.2 mi. W. of Indiana-Ohio State Line)

* New crossover location
Ref. Mkr.  I-94 Location Description

LaPorte District

16.3*  1.2 mi. E. of SR 51
18.0*  0.9 mi. W. of SR 249
19.9*  1.1 mi. E. of SR 249
21.7  0.6 mi. W. of US 20
23.1  0.8 mi. E. of US 20
25.1  0.9 mi. W. of SR 49
28.5  2.5 mi. E. of SR 49 (0.2 mi. W. EB Weigh Station)
31.5  3.0 mi. W. of US 421 (2.5 mi. E. of EB Weigh Station)
33.8  0.8 mi. W. of US 421
35.3*  0.7 mi. E. of US 421
38.8  1.1 mi. W. of US 20
41.3*  1.4 mi. E. of US 20 (1.5 mi. W. of Michigan City Rest Area)
43.7*  3.8 mi. E. of US 20 (0.5 mi. E. of Michigan City Rest Area)
45.8*  Indiana-Michigan State Line

* New crossover location

INTERSTATE-ROUTE CROSISOVERS, I-94

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* New crossover location

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**INTERSTATE-ROUTE CROSSEORS**

Three-Digit-Numbered Routes

*Figure 54-6B(100)*
CHAPTER 55

Geometric Design of Existing Non-Freeway (3R)

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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CHAPTER 55

GEOMETRIC DESIGN OF EXISTING NON-FREeway (3R)

55-1.0 INTRODUCTION

Section 40-6.0 identifies project scopes of work as follows:

1. new construction;
2. complete reconstruction, freeway;
3. partial reconstruction, freeway (4R);
4. reconstruction, non-freeway (4R);
5. 3R project, non-freeway;
6. 3R project, freeway;
7. partial 3R project, non-freeway;
8. high-accident location improvement, non-freeway; and
9. traffic-control-devices project.

Chapter 53 provides tables of geometric design criteria which apply to a new construction or reconstruction project. Chapters 40 through 52 provide design concepts and criteria which are directly applicable to new construction or reconstruction. For this type of project, the designer has the liberty of designing the highway to satisfy the most desirable and stringent criteria practical.

The geometric design of a project on an existing highway is viewed from a different perspective. This type of project is often initiated for reasons other than geometric design deficiencies (e.g., pavement deterioration, bridge replacement), and it often must be designed within restrictive right of way, and financial or environmental constraints. Therefore, the design criteria for new construction are often not attainable without major and, frequently, unacceptable adverse impacts. At the same time, however, the Department must use the opportunity to make cost-effective, practical improvements to the geometric design of an existing highway or street.

For these reasons, INDOT has adopted different limits for geometric design criteria for a project on an existing highway which are often lower than the values for new construction. The criteria for an existing highway are based on a sound, engineering assessment of the underlying
principles behind geometric design and on how the criteria for new construction can be modified to apply to an existing highway.

This chapter provides the Department’s criteria for a 3R non-freeway project. These criteria balance the many competing and often conflicting objectives. The objectives include improving an existing highway, minimizing the adverse impacts of highway construction on an existing highway, and improving the greatest longitudinal distance within the available funds for capital improvements. Where the 3R project scope of work is selected, costly work (e.g., bridge reconstruction or replacement, alignment improvements), which has a long service life and can be incorporated into a future 4R project, should desirably be constructed to satisfy 4R design criteria as part of the 3R project.

55-2.0 GENERAL REQUIREMENTS

55-2.01 Applicability

55-2.01(01) 3R Scope-of-Work Definition

A 3R project (rehabilitation, restoration, and resurfacing) on an existing non-freeway is intended to extend the service life of the existing facility and to enhance highway safety. A 3R project should make cost-effective improvements to the existing geometrics, where practical. This type of work on the mainline or at an intersection is on the existing alignment. Minimal right-of-way acquisition is required. Improvements for a 3R non-freeway project can include a combination of the following:

1. pavement resurfacing or rehabilitation or a limited amount of pavement reconstruction (30% or less of the traveled-way area);

2. bridge rehabilitation or replacement;

3. lane or shoulder widening;

4. upgrading the structural strength of shoulders;

5. flattening an occasional horizontal or vertical curve;

6. adjustments to the roadside clear zone;

7. flattening side slopes;
8. converting an existing median to a 2-way left-turn lane (TWLTL);  
9. adding a climbing lane;  
10. converting an uncurbed urban street into a curbed street;  
11. revising the location, spacing, or design of existing drives along the mainline;  
12. adding or removing a parking lane;  
13. bridge widening and associated substructure work to accommodate the widening;  
14. bridge-railing upgrading or replacement;  
15. bridge-deck overlay;  
16. work to preserve the bridge substructure;  
17. adding a sidewalk;  
18. relocating utility poles;  
19. upgrading guardrail or other safety appurtenances to satisfy certain criteria;  
20. other geometric or safety improvements to an existing bridge within the project limits;  
21. drainage improvements;  
22. increasing vertical clearance at an underpass;  
23. intersection improvement (e.g., adding turn lanes, flattening turning radii, channelization, sight-distance improvements, etc.);  
24. adding a new or upgrading an existing traffic signal; or  
25. other spot improvements.

Specifically related to the level of pavement improvement, the following definitions apply.
1. **Resurfacing.** Resurfacing consists of the placement of additional surface material over the existing restored or rehabilitated roadway or structure to improve serviceability or to provide additional strength.

2. **Restoration or Rehabilitation.** Restoration or rehabilitation is defined as work required to return the existing pavement to a condition of adequate structural support or to a condition adequate for the placement of an additional stage of construction. This may include milling the existing pavement.

### 55-2.01(02) National Highway System (NHS) Project

For long-range transportation planning purposes, INDOT has evaluated the State highway system to determine which routes warrant reconstruction, or 4R work, and which routes warrant a 3R-type improvement. Figure 55-2A shows a map of the State highway system which indicates 3R and 4R routes. The following will apply to the use of Figure 55-2A for such routes on the NHS.

1. **General.** The factors which will determine if a project should be classified as 3R or 4R are as follows:
   
a. If 70% or more of the existing pavement area of the traveled way can be retained and resurfaced, the project may be classified as 3R. If not, the project is classified as a 4R project.

b. An assessment of the level of service (LOS) for the 10-year traffic volume projection, which is based upon the expected service life of the pavement, can be used to determine if the project is 3R or 4R.

Other factors should also be considered when making the project scope of work determination (e.g., accident rates).

2. **4R Non-Freeway Route.** The Production Management Division’s Office of Environmental Services, or the local jurisdictional agency will determine the LOS for the 10-year traffic volume projection based on the discussion in Section 40-2.0. If this is LOS of D or better, it will be acceptable to design the project using the 3R geometric design criteria described in this chapter. If the projected LOS will not satisfy LOS of D, the facility will be designed according to the criteria for new construction or reconstruction. Each bridge replacement, bridge deck replacement, or bridge-widening should be designed to satisfy new construction, or 4R, criteria.
3. **3R Non-Freeway Route.** The project will be designed according to the 3R geometric design criteria described in this chapter. However, consideration could be given to using the 4R criteria.

4. **Combination Project.** Where a project will include both 3R and 4R work, the overall project scope-of-work classification should be based on the predominant type of work.

For example, a 6-mi resurfacing project which includes the replacement of one of the mainline bridges to 4R criteria will be classified as a 3R project, unless the bridge is considered to be a major structure and its replacement cost is equal to or greater than that of the 3R roadway work.

**55-2.01(03) Non-NHS Project**

The project scope-of-work definitions in Section 40-6.01 and Figure 55-2A, 3R/4R Systems, are intended only as general guidance for a non-NHS project. The decision on classifying a project that is not on the NHS should be made based on the future plans of the jurisdictional highway agency for the entire road between logical termini for the foreseeable future (20 years). All future plans for a road must consider current and projected traffic volumes, anticipated land use, and accident experience. The following provides examples of applying this concept to a non-NHS project.

1. **Example 1.** Approximately 60% of the pavement on a 6-mi section of a county road will be replaced. The remainder of the pavement is in reasonably good condition and only requires milling and resurfacing. The 6-mi section is part of a 30-mi county road which is the main highway between two small towns. The existing road has a LOS of A, and it is anticipated to provide a LOS of B based on 20-year projected traffic volume. There is no adverse accident experience for the past three years. Based on this information, a highway agency could decide to designate the 3R classification and construct the road to 3R design criteria. This is acceptable even though more than 30% of the pavement is being completely replaced.

2. **Example 2.** Approximately 40% of the pavement on a 6-mi section of county road will be replaced. The remainder of the segment will be resurfaced. This segment of road is part of a 25-mi county road which connects two small towns. This county road is located approximately 20 mi from a major metropolitan area. It is anticipated that, within the next 20 years, there will be considerable residential and commercial development adjacent to this stretch of county road because of its proximity to the rapidly expanding metropolitan area. The current LOS is B, but projected traffic volume indicates that the
LOS will drop to D in 10 years and to F in 20 years. For this situation, the highway agency has two options. It can design the project to 3R criteria for the present and, then, undertake a 4R project in 10 years when the pavement will likely be in need of major work. Its second option is to construct the project to 4R criteria now to satisfy future traffic demands.

3. **Example 3.** A 6-mi section of highway, which is located on INDOT’s 3R highway system, requires complete pavement replacement because of poor drainage. The Central Office has rechecked the status of this highway with the district office and verified that there are no plans for work on the remainder of this route in the future (20 years) except for 3R-type work. The current LOS is B, and it is anticipated to remain at B for the next 20 years. There is no adverse accident experience and no anticipated major land development along the route. INDOT can decide to only construct the project to 3R design criteria, though all of the pavement is being replaced.

4. **Example 4.** A 200-ft-long bridge on the State’s 3R system requires complete replacement. There are sharp horizontal curves on each end of the bridge where numerous accidents have occurred during the last three years. It has been decided to correct the poor alignment on the bridge approaches and to construct the approaches and bridge on a new location. The total length of the project is 1.5 mi. The Central Office has discussed the status of this road with the district office and both agreed that it should remain on the 3R system. The current LOS is B, and it is estimated that the LOS will be C in 20 years. There are no plans except to perform 3R-type work to the remainder of the road for the future (20 years). For this situation, INDOT can decide to construct the entire project to 3R design criteria.

5. **Example 5.** A 6-mi segment of a route on INDOT’s 3R system requires replacing 20% of the pavement and resurfacing the remaining 80%. The current LOS is D and will deteriorate to E in 5 years. There is rapid residential, commercial, and industrial development in the area. Both the Central and district offices agree that the entire route was properly classified as a 3R route. However, this one 6-mi segment is an exception because rapid growth adjacent to this segment is expected to occur. The appropriate solution in this situation is to upgrade the facility to accommodate anticipated traffic demand for the next 20 years and to design the project to 4R design criteria.
55-2.01(04) Procedures

For an INDOT project, the project scope of work is selected based on the following procedure.

1. The district office initially identifies the project scope.

2. The project is programmed based on the project scope determined by the district.

3. The Production Management Division’s Office of Environmental Services will make the final decision on the scope of work. However, for an Interstate-system project which has an estimated construction cost exceeding $1 million, FHWA will meet with representatives of the Office of Environmental Services to cooperatively agree on the project classification. This will occur as early in the project-scoping process as possible so that FHWA may have input on each project which is classified as 4R. The meeting will be held as soon as an initial concept for the project design has been developed.

4. The Production Management Division, during project design, may re-evaluate the project scope and request the Office of Environmental Services to modify the scope of work.

For a Federal-aid project not on the State highway system, the project scope of work determination will be based on the future plans of the local agency for improvements to its local road or street system. The philosophy described in Section 55-2.01(02) Item 2 for a 4R non-freeway State route should also be applied to a local project. The local agency must submit a letter to the Planning Division to document the local agency's plans on that facility in the foreseeable future. If the project is on the Interstate system and the estimated construction costs exceed $1 million, the Planning Division will schedule a meeting with the local agency and the FHWA to determine the project’s classification (3R or 4R). This meeting should occur early in the scoping process so that the FHWA may have input on each project that is classified as 4R.

55-2.02 Background

The 1976 Federal-aid Highway Act made it possible for the Department and local agencies to use Federal funds to extend the service life for the maximum number of centerline miles possible for the total highway system. On June 10, 1982, the FHWA issued its Final Rule entitled Design Standards for Highways; Resurfacing, Restoration and Rehabilitation of Streets and Highways Other Than Freeways. This rule modified 23CFR Part 625.4 to adopt a flexible approach to the geometric design of a 3R non-freeway project. Part 625.4 was modified again on March 31, 1983, to explicitly state that one objective of a 3R project is to enhance highway safety. In the rule, FHWA determined that it was not practical to adopt 3R design criteria for nationwide
application. Instead, each State was permitted to develop its own criteria or procedures for the design of a 3R project. This approach is in contrast to the application of criteria for new construction and reconstruction, for which the AASHTO *A Policy on Geometric Design of Highways and Streets* provides nationwide criteria for application. The flexible approach for 3R work permits Indiana to tailor its design criteria for its 3R program consistent with the conditions which prevail within the State. A highway for which geometrics were established some time ago is still capable of providing useful transportation service. Minor improvements will most often make such a highway serviceable for many more years.

In 1987, the Transportation Research Board (TRB) published Special Report 214, *SR214 Designing Safer Roads; Practices for Resurfacing, Restoration and Rehabilitation*. The objective of the TRB study was to examine the safety cost-effectiveness of highway geometric design criteria and to recommend minimum design criteria for a 3R project on a non-freeway. See *SR214* for more discussion.

INDOT has developed its own criteria for the geometric design of a 3R non-freeway project. Its objectives in developing these criteria may be summarized as follows:

1. extend the service life of the existing facility and to return its features to a condition of structural or functional adequacy;

2. incorporate highway safety enhancements, where judged to be cost effective; and

3. incorporate cost-effective, practical improvements to the geometric design of the existing facility.

55-2.03 Geometric-Design Approach

The Department’s approach to the geometric design of a 3R non-freeway project is to adopt, where justifiable, a revised set of numerical criteria. The design criteria throughout the other *Manual* chapters provide the frame of reference for the 3R criteria. The following summarizes the approach which has been used.

1. **Design Speed.** As discussed in Section 55-4.01, the design speed will be based on the existing posted or legal speed limit. The selected 3R design speed will then be used to evaluate all geometric design features of the existing highway which are based on speed (e.g., horizontal and vertical curvature).
2. **Cross-Section Width.** The criteria shown in Chapter 53 for new construction or reconstruction have been evaluated relative to the constraints of a 3R project. Where justifiable, the cross-section width criteria have been reduced. Where a range of values is provided in the Chapter 53 figures, the upper values have been incorporated into the 3R criteria to provide a desirable objective. This provides an expanded range of acceptable values for application on a 3R project. See Section 55-4.05 for additional discussion on cross-section width.

3. **Other Design Criteria.** Part V includes other proper geometric design techniques. These criteria are obviously applicable to new construction or reconstruction. For a 3R project, these criteria have been evaluated and a judgment has been made on their proper application to a 3R project. Unless stated otherwise in this chapter, the criteria in other chapters applicable to a 3R project should be incorporated if practical.

4. **Evaluation.** Available data, e.g., accident experience, should be evaluated when determining the geometric design of a 3R project. The following section discusses 3R project evaluation in more detail.

### 55-2.04 3R Project Evaluation

Sections 55-3.0 to 55-7.0 provide the specific geometric-design and roadside-safety criteria which will be used to determine the design of a 3R project. In addition, other factors must be considered in a 3R project design. Applicable evaluations should be conducted as may be deemed necessary. These evaluations are discussed below.

1. **Accident Experience.** The historical accident data within the project limits will be evaluated. This is the most critical element of 3R project evaluation to determine the appropriate level of geometric and safety improvement. Accident data is available from the Planning Division’s Office of Safety and Mobility. Section 55-8.0 further describes the Department's accident-analysis procedures.

2. **Existing Geometrics.** The designer will review the as-built plans and combine this review with the field review and field survey to determine the existing geometrics within the project limits. This includes lane and shoulder widths, horizontal and vertical alignment, intersection geometrics, and roadside-safety design.

3. **Speed Studies.** The designer will make the initial determination if a speed study is required for project design. The speed study should be conducted before the field review.
The speed study will be conducted by the district for an INDOT project, or by the local public agency or its consultant for a local-agency project.

4. Physical Constraints. The physical constraints within the limits of the 3R project will often determine what geometric improvements are practical and cost-effective. These include topography, adjacent development, available right-of-way, utilities, and environmental constraints (e.g., wetlands).

5. Field Review. The designer will conduct a thorough field review of the proposed 3R project. Other personnel should attend the field review as appropriate, including personnel from traffic, maintenance, construction, local agencies, etc. The objective of the field review should be to identify potential safety hazards and potential safety improvements to the facility.

6. Pavement Condition. A 3R project is sometimes programmed because of a significant deterioration of the existing pavement structure, including subbase, base, and surface courses. The extent of deterioration will determine the necessary level of pavement improvements. This decision will also influence the extent of practical geometric improvements. For a road to be eligible for resurfacing, the pavement should exhibit one or more of the following conditions such that a timely resurfacing is needed to prevent more serious deterioration.

   a. alligator cracking.
   b. bleeding.
   c. block (cracking).
   d. bumps (upheaval).
   e. corrugation.
   f. depression and rutting.
   g. edge cracking.
   h. longitudinal and transverse cracking.
   i. patching or utility cut.
   j. polished aggregate;
   k. potholes.
   l. slippage cracking; or
   m. weathering and raveling.

The proposed pavement improvement will be based on the design-year traffic volume. The design year is 10 years after construction for a resurface project, or 20 years after construction for a pavement-replacement project. The pavement surface will be designed to incorporate skid resistance.
7. **Structures.** A 3R project may include bridges and culverts within the project limits or a 3R project may be a bridge improvement. Each bridge or culvert should be evaluated for possible structural improvements which may include the following:

a. increasing the structural loading capacity;
b. improving the roadside safety (e.g., upgrading the bridge railings);
c. improving the horizontal and vertical alignments;
d. widening the structure; or
e. increasing the facility’s hydraulic capacity.

8. **Geometric Design of Adjacent Highway Sections.** The designer should examine the geometric features and operating speeds of highway sections adjacent to the 3R project. This will include investigating whether or not highway improvements are in the planning stages. The 3R project should provide design continuity with the adjacent sections. This involves a consideration of factors such as driver expectancy, geometric design consistency, and proper transitions between sections of different geometric designs.

9. **Early Coordination for Right-of-Way Acquisition or Utility Accommodation.** Field reviews and accident or speed studies may indicate the need for selective safety improvements which will require right-of-way purchases. Right-of-way acquisition should be initiated as early as feasible.

Utility relocation and accommodation is frequently encountered. Therefore, early coordination with utility companies is essential.

10. **Traffic Operations.** The designer should evaluate existing traffic operations to determine where improvements can be reasonably implemented (e.g., adding turn lane, removing a signal, adding additional lane through an intersection). The designer should also review the effect construction will have on traffic operations. This may require reprogramming signals, implementing a phased construction plan, etc. Part VIII provides additional information on traffic management through a construction zone.

11. **Maintenance and Protection of Traffic.** A 3R project can only occur on an existing highway. Therefore, maintenance and protection of traffic during construction will be an important consideration in 3R project development. See Part VIII for criteria on the design of the work zone for traffic accommodation.

12. **Traffic-Control Devices.** All signing and pavement markings should be in accordance with Part VII and the *Manual on Uniform Traffic Control Devices* (MUTCD). The
district traffic office or the local agency is responsible for selecting and locating the traffic-control devices. However, the designer should work with the proper authority to identify possible geometric and safety deficiencies which will remain in place (i.e., no improvement will be made). These may include the following:

a. narrow bridge;
b. horizontal or vertical curve which does not satisfy the 3R criteria; or
c. roadside hazard within the obstruction-free zone.

The proper authority will then determine if additional signing, traffic-control devices, or delineation treatments are warranted.

13. Documentation of Design Process. The designer should prepare an Engineer’s Report for an INDOT-route project or a Safety and Design Report for a local-agency project. The report should include the following:

a. existing geometric and roadside features, traffic volume and speed, and accident history;
b. applicable minimum design criteria;
c. specific safety problems or concerns raised by a review of accident data by a field inspection or by the public;
d. design options for correcting safety problems and the cost, safety, or other relevant impacts of these options;
e. proposed exceptions to applicable design criteria and the rationale to support the exceptions; and
f. the recommended design proposal.

The designer must also prepare a list of potential design exceptions, which must be fully documented in accordance with Section 40-8.0.
55-3.0 GEOMETRIC DESIGN CRITERIA [REV. JUL 2014]

Figures 55-3A through 55-3H provide the Department’s criteria for the design of a 3R non-freeway project, either in a rural or urban area. See Section 55-4.01(03) for information regarding adherence to design criteria.

The criteria are assigned the figure numbers and are titled as follows:

- **55-3A** Geometric Design Criteria for Rural Arterial, 3R Project
- **55-3B** Geometric Design Criteria for Rural Collector, State Route, 3R Project
- **55-3C** Geometric Design Criteria for Rural Collector, Local-Agency Route, 3R Project
- **55-3D** Geometric Design Criteria for Rural Local Road, 3R Project
- **55-3E** Geometric Design Criteria for Urban Arterial, Four or More Lanes, 3R Project
- **55-3F** Geometric Design Criteria for Urban Arterial, Two Lanes, 3R Project
- **55-3G** Geometric Design Criteria for Urban Collector, 3R Project
- **55-3H** Geometric Design Criteria for Urban Local Street, 3R Project

The designer should consider the following in the use of the figures:

1. **Project Scope of Work.** The Department has adopted separate criteria for the geometric design of a new construction or reconstruction project. See Chapter 53. Chapter 40 provides definitions for a non-freeway project scope of work, which will determine which set of criteria to use for project design.

2. **Functional Classification.** The selection of design values depends on the functional classification of the highway facility. This is discussed in Section 40-1.01. Functional classification maps for all public roads in the State are available from the Planning Division.

3. **Urban Design Subcategories.** Within an urbanized or urban area, the selection of design values depends on the design subcategory of the facility. Separate criteria are provided for suburban, intermediate, and built-up subcategories. These classifications are defined as follows.

   a. **Suburban.** This type of area is located at the fringe of an urbanized or small urban area. The predominant character of the surrounding environment is residential, but it may include a considerable number of commercial establishments, especially strip development along a suburban arterial. There may also be a few industrial parks. On a suburban road or street, motorists have a significant degree of freedom but, nonetheless, they must also devote some of their attention to...
entering and exiting vehicles. Roadside development is characterized by low to moderate density. Pedestrian activity may or may not be a significant design factor. Right of way is often available for roadway improvements.

A local or collector street is located in a residential area, but may also serve a commercial area. The posted speed limit ranges between 30 and 50 mph. The majority of intersections will have stop or yield control, but there will be an occasional traffic signal. A suburban arterial will have strip commercial development and perhaps a few residential properties. The posted speed limit ranges between 35 and 55 mph. There will usually be a few signalized intersections along the arterial.

b. Intermediate. As the name implies, an intermediate area is between a suburban and a built-up area. The surrounding environment may be either residential, commercial, or industrial or a combination of these. The extent of roadside development will have a significant impact on the selected speeds of motorists. The increasing frequency of intersections is also a major control on average speeds. Pedestrian activity has now become a significant design consideration, and sidewalks and crosswalks at intersections are common. The available right of way will restrict the practical extent of roadway improvements.

A local or collector street has posted speed-limit ranging between 30 and 45 mph. The frequency of signalized intersections has increased substantially if compared to a suburban area. An arterial will have intensive commercial development along its roadside. The posted speed limit ranges between 35 and 50 mph. Such an arterial has several signalized intersections per mile.

c. Built-up. This type of area refers to the central business district within an urbanized or small urban area. The roadside development has a high density and is often commercial. However, a substantial number of roads and streets pass through a high-density environment (e.g. apartment complexes, row houses). Access to property is the primary function of the road network. Pedestrian considerations may be as important as vehicular considerations, especially at intersections. Right of way for roadway improvements is usually not available.

Because of the high density of development, the distinction between the functional classifications (local, collector, or arterial) becomes less important in considering signalization and speeds. The primary distinction among the three functional classifications is the relative traffic volumes and, therefore, the number
of lanes. As many as half the intersections may be signalized. The posted speed limits ranges between 25 and 35 mph.

See Section 40-1.01 for definitions of the functional classifications.

4. **Rural-Area Figures.** These do not provide design criteria for sub-categories. However, there are many rural facilities which pass through relatively built-up, but unincorporated, areas. It may be inappropriate to use the rural-area design criteria. The designer may, as an option, use the suburban criteria for a functional classification (e.g., arterial) in a relatively built-up rural area. Therefore, if the area is urban in character (e.g., a densely populated area with a grid-like street system), it may be appropriate to use the urban-area design criteria, though the facility is rural. This decision will be documented in the Engineer’s Report (see Chapter 7).

5. **Cross-Section Elements.** Some of the cross-section elements included in a figure (e.g., sidewalk width) are not automatically warranted in the project design. The value will apply only after the decision has been made to include the element in the highway cross section.

A 3R project should not be designed with a narrower roadway width than the existing facility. See Section 55-4.05.

6. **Indiana Design Manual Section References.** The figures are intended to provide a concise listing of design values for easy use. However, the designer should review the Manual section references for greater insight into the design elements.

7. **Footnotes.** The figures include many footnotes, which are identified by a number in parentheses, e.g., (6). The information in the footnotes is critical to the proper use of the figure.

8. **Controlling Design Criteria.** An asterisk indicates each controlling design criterion which, if not satisfied, requires a Level One design exception. The discussion in Section 40-8.0 on design exceptions applies equally to the geometric design of a 3R project. However, the designer will evaluate the proposed design against the criteria described in this chapter.
55-4.0 GEOMETRIC DESIGN

55-4.01 Design Controls

55-4.01(01) Traffic-Volume Analysis

The following traffic-volume controls will apply.

1. **Design Year.** Pavement resurfacing should be designed using a 10-year design life. Pavement replacement and all other elements of the facility should have a design life of 20 years beyond the expected construction date.

2. **Level of Service (LOS).** The appropriate figure in the 55-3 series provides the desirable and minimum LOS criteria.

3. **Traffic Data.** The designer should obtain, from the Production Management Division’s Office of Environmental Services, the traffic data necessary to determine the level of improvement. At a minimum, this will include current and future (10 and 20 years) AADT, DHV, percent of trucks and buses, turning movements at intersections, accident data for the most recent 3-year period, and known future traffic impact.

4. **Capacity Analysis.** The analytical techniques described in the *Highway Capacity Manual* should be used to conduct the capacity analysis.

55-4.01(02) Design Speed

The existing posted or legal speed limit will most often be selected as the design speed. More specifically, the design speed should be the highest posted speed limit or legal speed limit existing on logical sections of the roadway consistent with the expectations for that section of roadway and future improvement plans. Logical sections will be based on land use and topography. If a road is not posted, it is desirable to perform an engineering study to determine an appropriate posted speed limit.

If the facility is posted, it may be appropriate to perform an engineering study if there is sufficient reason to believe that the existing posted speed limit may change after project completion. The designer may request, and the district traffic office or local jurisdiction may determine, that a speed study within the project limits is necessary to establish a 3R design speed.
Section 40-3.02 discusses the relationship between the project design speed and the legal speed limit. The Section also provides the legal speed limits from the State statutes which apply to all public roads.

In summary, the selection of a 3R project design speed will be one of the following:

1. the existing posted speed limit;
2. the legal speed limit on a non-posted facility;
3. a revised posted speed limit or the anticipated posted speed limit on a currently non-posted facility; based on the results of a speed study; or
4. a design speed which is higher than the posted or regulatory speed limit, where deemed to be appropriate.

55-4.01(03) Adherence to Design Criteria [Rev. Jul. 2014]

The discussion in Section 40-8.0 regarding design exceptions applies equally to the geometric design of a 3R project. The values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the Green Book) may be used as minimum values if they are lower than similar values shown herein except as follows.

1. The Green Book minimum values may not be used to supersede State or Federal code requirements, e.g. National Truck Network, American with Disabilities Act (ADA).
2. Vertical clearance requirements for new and replaced bridges, sign trusses, and pedestrian structures must include an additional 6” for consideration of future resurfacing.
3. The minimum bridge clear roadway width requirements in this chapter apply.

When the Green Book minimum values or exceptions as noted above for Level One controlling criteria are not met, a design exception is required. See Section 40-8.04(01).

55-4.02 Sight Distance

The criteria described in Chapter 42 regarding sight distance apply equally to a 3R project. However, the application of the sight-distance criteria to each individual highway element (e.g.,
vertical curve) on a 3R project will differ from that applied to a new construction or reconstruction project. These are discussed at the applicable locations elsewhere in this chapter.

55-4.03 Horizontal Alignment

Engineering judgment or a cost-effectiveness evaluation will ultimately reveal the need for improvements to the horizontal alignment. Improvements to the horizontal alignment should be considered if a specific problem is identified. Examples include the following:

1. a disproportionate run-off-the-road accident rate at a curve site;
2. a disproportionate number of multi-vehicle accidents at a curve site; or
3. the presence of an adverse accident history at an intersection within a curve.

The evaluation of potential improvements will include a consideration of traffic volume, truck volume, right-of-way and utility impacts, environmental impacts, driver expectancy, construction costs, etc.

55-4.03(01) Minimum Horizontal-Curve Radius

The designer should determine the Computed Existing Design Speed (CEDS) of the each curve radius within the 3R project limits. To determine the CEDS, the designer should determine the applicable maximum superelevation rate for the project location. For a rural highway or an urban facility where \( V \geq 50 \text{ mph} \), an \( e_{\text{max}} \) of 8% should be used (see Figure 43-3A). For an urban facility where \( V \leq 45 \text{ mph} \), an \( e_{\text{max}} \) up to 6% may be used (see Figure 43-3C). An existing horizontal curve may be retained if the conditions exist as follows:

1. the accident data does not indicate a problem at the curve site;
2. the CEDS is not more than 15 mph below the 3R design speed; and
3. the AADT is not greater than 750 vehicles per day.

The existing radius will be retained on a curve where the above conditions are satisfied (i.e., the curve need not be evaluated). However, proper signs and markings may be necessary to inform the motorist of non-conforming criteria. If the above conditions are not satisfied on an existing horizontal curve, a safety benefit/cost study (B/C) should be conducted to determine if the proposed
correction will be cost effective. Chapter 50 describes the Department’s procedures for conducting a benefit/cost analysis. If the B/C ratio is less than 1.0, the existing horizontal curve may be retained. Where the B/C ratio is greater than or equal to 1.0 and it is decided to reconstruct the curve to satisfy the minimum-radius criteria, the curve should desirably be reconstructed to satisfy all horizontal-alignment requirements for new construction or reconstruction (e.g., superelevation rate, superelevation transition length, distribution of superelevation between tangent and curve). See Chapter 43. If reconstruction is shown to be cost effective and it is decided not to undertake the work, it will be necessary to request a Level One design exception.

55-4.03(02) Superelevation

On a horizontal curve where the existing radius will be retained, it may be warranted to make improvements to the superelevation. The following will apply.

1. **General.** The most desirable objective is to improve the horizontal curve to satisfy all superelevation criteria shown in Section 43-3.0.

2. **Rate.** Where the CEDS is less than the design speed, the superelevation rate should be increased to provide the design speed, up to a maximum of 8% (rural) or 6% (urban).

   In an urban area, it may be appropriate to remove or reduce the existing superelevation if the design speed of the revised curve will equal or exceed the project design speed (see Section 43-3.02). This may be advantageous to better satisfy the roadside development or drainage conditions, or to provide better operations at an at-grade intersection.

3. **Transition-Length Distribution.** The superelevation transition length will be distributed by placing 60% to 70% on the tangent and the remainder on the horizontal curve. However, where this is not practical, a reduction to a 50% to 50% distribution is acceptable.

4. **Shoulder Superelevation.** The travelway-to-shoulder rollover break is placed at the edge of travelway on the outside of a horizontal curve. However, where a paved shoulder of 4 ft or narrower is used, the break should occur at the outside edge of the paved shoulder.

55-4.03(03) Reverse Curves

It may be acceptable to leave reverse curves in place if the PT and PC are coincident. To determine if improvements are warranted, existing combined reverse curves should be evaluated using the criteria in Section 43-3.07, and for each individual curve, Sections 55-4.03(01) and 55-4.03(02). An
evaluation of the accident history should be made for existing reverse curves (e.g., multi-vehicle accidents).

55-4.03(04) Broken-Back Curves

For existing broken-back curves, the designer should, if practical, eliminate the curves and combine them into a single, continuous horizontal curve, especially where an evaluation of the accident history indicates a problem.

55-4.03(05) Curves in Series

The alignment of a segment of a roadway often consists of a series of reverse curves or curves connected by short tangents. A succession of curves may be analyzed as a unit rather than as individual curves, applying the criteria described in Section 55-4.03(01).

1. The first substandard curve in a series should be analyzed individually as this change in alignment prepares the driver for the remaining curves in the series.

2. An intermediate curve in a series of substandard curves that is significantly worse than the others in the series should also be analyzed individually.

3. These controlling curves can be used to determine the safety or other mitigation measures to apply throughout the series.

3. Where improvements are considered to curves in a series, the effect on the series of curves as a whole should be evaluated.

55-4.03(06) Shoulder Treatment

On a facility with relatively sharp horizontal curves and truck volume greater than 500 per day, a full-structural strength shoulder should be provided on both sides of a sharp horizontal curve in place of pavement widening. The following will apply.

1. **Strengthened Length.** The strengthened shoulder should be available from the beginning of the superelevation transition before the curve to the end of the transition beyond the curve.
2. **Asphalt Traveled Way.** The pavement structure of the strengthened shoulder should match that of the traveled way.

3. **Concrete Traveled Way with Asphalt Shoulder.** The Office of Pavement Engineering will determine the pavement structure of the strengthened shoulder.

4. **Concrete Traveled Way with Concrete Shoulder.** The concrete-shoulder thickness should match that of the traveled way.

55-4.03(07) **Horizontal Sight Distance**

Section 43-4.0 provides criteria for determining if the applicable sight distance is available at a horizontal curve. If an existing longitudinal barrier interferes with the line of sight at a horizontal curve, the designer should review practical alternatives to alleviate the problem, such as eliminating the hazard that requires the barrier or offset the barrier further from the travel lane. If it is determined to leave the barrier in its existing location, it will be necessary to seek a design exception for the stopping sight distance.

55-4.03(08) **Traffic-Control Devices**

For an existing horizontal curve to remain as such, traffic-control devices that may be considered to improve motorist safety and comfort include the following:

1. signing (e.g., advance warning, chevron);
2. raised pavement markers; or
3. reflective marker posts or delineators.

Part VII and the MUTCD discuss the selection and installation of traffic-control devices in more detail.

55-4.04 **Vertical Alignment**

55-4.04(01) **Grades**

The appropriate figure in the 55-3 series provides the Department’s criteria for maximum and minimum grades. The maximum grade is 1% steeper than that for new construction or reconstruction on a rural arterial, or 2% steeper for another type of facility. Improvements to an
existing grade should be considered if a specific problem is identified (e.g., head-on accidents due to improper passing maneuvers, significant speed reduction for trucks).

55-4.04(02) Climbing Lane

The warrants for a climbing lane shown in Section 44-2.0 are also applicable to a 3R project. The following will apply to the design of a climbing lane.

1. **New.** The criteria shown in Section 44-2.0 should be used.

2. **Existing.** Desirably, the criteria shown in Section 44-2.0 should be used. However, existing lane and shoulder widths may be retained if there is no adverse accident history that can be related to the narrower width.

55-4.04(03) Crest Vertical Curve

Existing crest vertical curves will most often be incorporated into a 3R project. An existing crest vertical curve may be retained if the conditions exist as follows:

1. there is no history of accidents related to the vertical curve (e.g., rear-end accidents);

2. the crest does not hide major hazards from view such as an intersection, sharp horizontal curve, or a narrow bridge;

3. the CEDS of the existing crest (based on minimum sight distance for a passenger car) is not more than 20 mph below the 3R-project design speed using a 2-ft object height; and

4. the design-year AADT is not greater than 1500.

If an existing crest vertical curve does not satisfy all of the criteria listed in Items 1 through 4 above, such that reconstruction may be warranted, a benefit/cost (B/C) study should be conducted to determine if the proposed correction will be cost effective. Chapter 50 provides the Department’s procedures for conducting a benefit/cost analysis. If the B/C ratio is less than 1.0, then the existing vertical curve can be retained. Where the B/C ratio is greater than or equal to 1.0 and it is decided to reconstruct the vertical curve, it should be designed using the criteria for new construction/reconstruction (see Section 44-3.0). If reconstruction is shown to be cost-effective and it is decided not to undertake the work, it will be necessary to request a Level One design exception.
55-4.04(04) Sag Vertical Curve

Section 44-3.0 provides the Department’s criteria for the design of a sag vertical curve for new construction or reconstruction. These criteria are based on designing the sag to allow the vehicular headlights to illuminate the pavement for a distance equal to the stopping sight distance for a passenger car. An existing sag vertical curve may be evaluated using the comfort criteria shown in Figure 55-4A, K Value for Sag Vertical Curve (Comfort Criteria - 3R Project).

The following options for evaluating a sag vertical curve are shown below in order from the most desirable to the least desirable.

1. Improve the sag vertical curve to the new construction or reconstruction criteria shown in Section 44-3.0 if it is cost effective to do so.

2. Improve the sag vertical curve to be in accordance with the K value for comfort criteria shown in Figure 55-4A. An existing sag vertical curve that can be improved by wedge and level up to 18 in. depth to be in accordance with the comfort criteria shown in Figure 55-4A, may be retained.

3. Reconstruct the sag vertical curve to an improved level, but not in full accordance with the comfort criteria.

4. Retain the existing sag vertical curve though it is not in accordance with the comfort criteria.

If an existing sag vertical curve does not satisfy the comfort criteria shown in Figure 55-4A, or there is a history of accidents related to the curve such that reconstruction may be warranted, a benefit/cost study should be conducted to determine if the proposed correction will be cost effective. Chapter 50 provides the Department’s procedures for conducting a benefit/cost analysis. If improvement in accordance with Section 44-3.0 is shown to be cost-effective and it is decided not to undertake the work, it will be necessary to request a Level One design exception.

55-4.04(05) Curves in Series

The vertical alignment of a segment of a roadway can consist of a series of sag and crest vertical curves or vertical curves connected by short grades. A succession of vertical curves may be analyzed as a unit rather than as individual curves, applying the criteria in Sections 55-4.04(03) and 55-4.04(04). Analysis procedures similar to Section 55-4.03(05) Items 1 through 4 should be followed.
55-4.04(06) Angle Point

It is acceptable to retain an existing angle point, with no vertical curve, of 0.5% algebraic difference for a crest situation, or 1.0% algebraic difference for a sag situation.

55-4.05 Cross-Section Elements

Chapters 45 and 53 provide the Department’s criteria for cross-section elements for a new construction or reconstruction project. The figures in Section 55-3.0 provide the cross-section criteria for a 3R project. The criteria were established as follows:

1. **Upper Limit.** The upper limit, or desirable, value in the range has been established as equal to the upper level for new-construction criteria. This still provides a desirable objective for the design of the cross-section elements.

2. **Lower Limit.** The lower limit, or minimum, value in the range has been established by considering the minimum acceptable width for the element from an operational and safety perspective. Consider what will be available for a practical improvement by also considering that it is better to improve a greater length of roadway to a lower level than to improve a shorter length of roadway to a higher level. All of these considerations are consistent with the overall objectives of the Department’s 3R program.

The width or steepness of the existing cross section should be evaluated against the criteria shown in the appropriate 55-3 series figure. If the existing width or steepness does not satisfy the minimum 3R criteria, the designer should consider widening or flattening the element. If the decision is made to widen or flatten the cross-section element, the designer should provide a design which at least satisfies the minimum 3R criteria. This will ordinarily be sufficient. However, if practical, it may be appropriate to widen or flatten the highway elements to satisfy the desirable 3R criteria.

The following summarizes the Department’s 3R criteria for cross-section elements.

55-4.05(01) Travel-Lane Width

A 3R project should include practical improvements to the existing lane widths, if needed. The designer should consider the following regarding trucks.
1. **Rural Arterial.** Each rural arterial is on the National Truck Network and should have 12-ft travel lanes. Section 40-1.05 provides additional information on the National Truck Network.

2. **Urban Arterial.** For each urban arterial on the National Truck Network, the right lane in each direction should be 12 ft. For an arterial of four or more lanes, the centerline of roadway should not be shifted to accommodate the 12-ft right lane. The additional pavement width should be obtained by widening on the outside only.

3. **Other Route.** For another type of route, a minimum of width of 11 ft should be provided, if there are more than 200 trucks per day in the design year.

**55-4.05(02) Shoulder Width**

A 3R project should include widening of the existing shoulders, if needed.

**55-4.05(03) Paved-Roadway Width**

The paved-roadway width should not be less than that of the existing facility.

**55-4.05(04) Lane and Shoulder Cross Slopes**

Shoulder cross slopes on a horizontal curve should be in accordance with Section 43-3.06. The low-side shoulder should desirably be sloped as described in Section 43-3.06(02). At a minimum, the same cross slope on the shoulder should be kept in a tangent section.

Restoring or improving the pavement cross slope is often cost effective, resulting in improved ride, safety, and drainage, and maintenance of roadway pavements.

**55-4.05(05) Parking Lanes**

For an urban-area project, the designer must evaluate the demand for, or the elimination of, on-street parking. Section 45-1.04 provides the Department’s policy for the removal or addition of on-street parking.
55-4.05(06) Curbs

The following will apply to the installation or retention of curbs.

1. **Types.** Where the work will disturb an existing curb, the curb is replaced in-kind.

2. **Height.** Pavement work may be included which does not affect the lateral location of existing curbs but will affect their finished height. The curb height, or the pavement section, should be considered for adjustment as follows:

   a. an analysis of the stormwater flow in the gutter indicates overtopping the curb for the design parameters (e.g., design-year frequency, ponding on roadway);

   b. the existing curb is deteriorated; or

   c. the curb height after construction will be less than 3 in.

3. **Safety Considerations.** On a facility with design speed of 50 mph or higher, existing curbs should be removed for safety considerations, if they are not needed for drainage.

55-4.05(07) Sidewalks

Where the work will disturb an existing sidewalk, the sidewalk is reconstructed or replaced in-kind, including curb ramps. Where a sidewalk does not currently exist, the need for a sidewalk will be determined as discussed in Section 45-1.06. Sidewalk construction and maintenance funding are dependent upon the project location. The following will apply.

1. **Town or Rural Area.** A new sidewalk constructed outside the town limits may be funded with State and Federal funds.

2. **City Limit.** For a sidewalk constructed within the corporate city limits with Federal funds, INDOT may elect to participate in the cost of constructing the sidewalk. For a non-Federally funded project, the city will be responsible for the costs of constructing the sidewalk. A reimbursement agreement will be required between the Department and the city prior to the project letting. The State will be responsible for the cost of right-of-way and grading required specifically for the sidewalk.

3. **Bridge.** Regardless of location, the total cost for sidewalks on a bridge may be funded with State and Federal funds.
Curb ramps should be provided at all pedestrian crosswalks within the project limits. See Section 51-1.0 and the INDOT *Standard Drawings* for additional information on accessibility requirements.

### 55-4.05(08) Median Width

The following will apply to median width.

1. **Existing Median.** An existing divided non-freeway may be improved as a 3R project. If so, the existing median width will be retained.

2. **Flush Median.** If the median width is 16 ft or less, the designer should consider using a continuous raised corrugated median. The INDOT *Standard Drawings* provide additional details for a corrugated median. For additional information on a flush median, see Section 45-2.02.

3. **Raised Median.** For additional information, see Section 45-2.02.

### 55-4.05(09) Fill or Cut Slopes

The following will apply to fill or cut slopes.

1. **No Roadway Widening.** Existing fill or cut slopes of 2:1 or flatter will be retained.

2. **Roadway Widening.** If the lanes or shoulders are widened, this will produce a steeper fill slope or ditch foreslope, assuming the toe of fill slope or toe of backslope remains in the same location. The roadside design should desirably be modified to provide a configuration which is the same as or flatter than the roadside cross section before the 3R project limits. At a minimum, the following will apply:

   a. **Embankment slope.** The use of a 3:1 slope should be considered. However, an effort should be made to construct up to a 6:1 slope at least within the obstruction-free zone where a 6:1 or flatter slope already exists, or where the length of the improvement is greater than 0.5 mi. See Section 55-5.0 for obstruction-free zone dimensions. If a steeper slope is required, a 2.5:1 slope should be considered before implementing a 2:1 slope. The slope behind the guardrail at a bridge corner should not be routinely steepened to 2:1 even though the slope may be completely protected by the guardrail. Locations or situations that may warrant a 2:1 slope are as follows:
(1) roadway widening that encroaches into a wetland;

(2) an area with restrictive or very costly right of way; or

(3) a slope at the end of a large culvert, bridge spillslope, or other location where it is desirable to protect the slope with riprap.

Where a 2:1 slope is specified, it should be protected with erosion control blankets. Capping soils suitable for growing vegetation should be provided.

The use of a 2:1 slope in a local-agency project will be at the discretion of the local agency.

Each location must be analyzed individually, and judgment should be used in selecting the slope rate.

b. Ditch. If right of way is available, the existing ditch line should be moved and the slopes flattened as much as practical. A drainage ditch in the obstruction-free zone should be regraded as much as practical to make it traversable for an errant vehicle. See Section 49-3.02 for information on traversable ditch.

c. Guardrail. Consideration should be given to obtaining a 3:1 slope in a fill to minimize the need for guardrail. An embankment should desirably be widened where guardrail will be installed as required by Section 55-5.0.

d. Embankment Stability. Sod or other stabilizing materials or methods should be provided wherever erosion may be considered to be a problem.

3. Roadside Safety. Upgrading the roadside safety is often a major objective. The designer should consider the safety benefits of flattening each fill or cut slope to eliminate guardrail and, at a minimum, to satisfy the criteria described in Item 2 above. An evaluation of run-off-the-road accidents will assist in the assessment (see Chapter 50). See Section 55-5.0 for more information regarding roadside-safety criteria.
55-4.05(10) Right of Way

Only minimal right-of-way acquisition should be required (e.g., lane and shoulder widening). More-extensive right-of-way involvement may be appropriate if, for example, a horizontal curve is flattened. Where practical, additional right-of-way should be secured to allow cost-effective geometric and roadside-safety improvements.

55-4.06 Intersection At-Grade

Chapter 46 provides criteria for the detailed design of an intersection at-grade for new construction or reconstruction. Where practical, these criteria apply to a 3R project and should be implemented. The following indicates where modifications to the intersection design criteria may be made.

55-4.06(01) General Design Controls

The criteria provided in Section 46-1.0 for intersection alignment, profile, design vehicle selection, etc., also apply to a 3R project, except as follows:

1. Intersection Alignment. Preferably, the angle of intersection should be within 20 deg of perpendicular. An existing angle of intersection of up to 30 deg may be retained if there are no operational problems or adverse accident history.

2. Y Intersection. Each existing Y intersection should be converted to a T intersection.

3. Design-Vehicle Selection. An existing intersection should be checked to determine if the suggested design-vehicle criteria shown in Figure 46-1E can be accommodated using the criteria shown in Section 55-4.06(02) for turning radius. An intersection which cannot accommodate the minimum design vehicle should be considered for reconstruction.

55-4.06(02) Turning Radius

Unless alerted by district personnel or where there is physical evidence of problems at an intersection such as tire tracks over curbs, broken curbs, or scraped utility poles, it should not be necessary to reconstruct the intersection to improve the turning radii design as part of the 3R project. However, once it has been determined to upgrade the intersection, the design should desirably be in accordance with Section 46-2.0. In an urban area, however, space limitations and existing curb radii
have a significant impact on selecting a practical design for a right-turning vehicle. The designer should consider the following when determining the appropriate right-turn treatment for an urban intersection.

1. **Inside Clearance.** The minimum inside clearance of the selected design vehicle may be zero; i.e., the inside tire track may touch the curb line or pavement edge.

2. **Encroachment.** Once the decision has been made to improve an intersection, the selected design vehicle’s path should be in accordance with the encroachment criteria discussed in Section 46-2.0. Under restricted conditions, an additional 1-ft encroachment is permitted for each functional classification.

3. **Sweep-Path.** The designer should review the existing or redesigned intersection with the turning templates to ensure that there are no obstacles in the sweep-path of the turning design vehicle.

4. **Minor Intersection.** At an intersection with at least one leg considered a minor road, a school bus, garbage truck, or fire truck should physically be able to make the turn onto the minor road.

The requirements regarding acceptable existing turning radius are as follows.

1. **Passenger Car.** A radius of 15 to 25 ft is adequate. This may be retained on an existing cross street as follows:
   a. intersection with a minor road where few trucks will be turning;
   b. intersection where the encroachment of a single-unit truck or a tractor-and-semitrailer combination onto adjacent lanes is tolerable; or
   c. intersection where a parking lane is present, it is restricted for a sufficient distance from the intersection, and it is used as a parking lane for a specified period each day.

2. **Single-Unit Truck.** An existing radius of at least 30 ft or a radius with taper offsets for this vehicle may be retained.

3. **Tractor-and-Semitrailer Combination or Bus.** At an intersection where these vehicles turn frequently, an existing radius of at least 40 ft or a radius with taper offsets may be retained.
55-0.06(03) Turn Lane

Section 46-4.0 provides warrants for a right- or left-turn lane and design requirements for an auxiliary turn lane. These should be satisfied if practical. However, the criteria for new construction or reconstruction may be impractical due to restricted site conditions. Specific examples of acceptable design criteria for an auxiliary turn lane are as follows.

1. **Shoulder.** An existing paved shoulder of sufficient width and pavement strength may be striped to indicate a separate right-turn lane at an intersection. If so, it may be necessary to rebuild or redesign the curb return to accommodate the selected design vehicle.

2. **Reduced Travel-Lane Width.** In an urban area, the width of the approaching travel lane may be reduced at a signalized intersection to provide a reasonable width for a turn lane. However, travel lanes should be at least 10 ft wide at the intersection and may be warranted to be wider if truck traffic turns must be accommodated.

3. **Width.** This may be narrower than that for new construction or reconstruction work.

4. **Length.** The length should desirably include the components for taper, deceleration, and storage as described in Section 46-4.02. These criteria may be impractical, particularly the length for the vehicular-deceleration component. However, the minimum length shown in Section 46-4.02 applies.

55-0.06(04) Intersection Sight Distance

Intersection sight distance should be in accordance with Section 46-10.0. The location of the eye should be 14.5 ft from the edge of the travel lane with respect to a stop-controlled intersection.

55-5.0 ROADSIDE SAFETY

Many of the improvements will have a positive effect on highway safety. In addition, a 3R project affords an opportunity to further enhance highway safety by accomplishing needed safety improvements at high-hazard locations and cost-effective adjustments or modifications to high-hazard features. Section 49-10.0 provides information on how to use ROADSIDE, a computer program which may be used to determine if roadside-safety improvements are cost effective. The following discussion offers roadside-safety criteria which apply specifically to a 3R project.
55-5.01 Analysis of Accident Data

The designer should obtain the accident history for the three-year period immediately prior to the year in which project design is initiated. The data may be summarized on the form included in Section 55-8.0 or in another convenient format.

The data should be analyzed to determine if there are any correctable accident patterns at a particular spot location of 1000 ft minimum length, intersection, or section of the highway. If a pattern exists, probable causes should be identified and appropriate safety enhancements included in the work. Each intersection or highway section which has an average of four or more accidents per year for the three-year period should be analyzed in accordance with the guidelines described in Section 55-8.0. This will require obtaining copies of the accident reports for these locations and possibly the preparation of collision diagrams. A short discussion of the probable causes and corrective action to be incorporated into the project for each intersection or highway section should be included in the Engineer’s Report for an INDOT project, or in the Safety and Design Report for a local public agency project. An intersection or highway section may be experiencing the types of accidents that are correctable by highway improvements. The analysis may reveal that there is no apparent safety enhancement that can be included in the project. If this situation exists, a short discussion should be included in the Report to document that each such intersection or highway section was reviewed.

A list of high-accident locations has been developed by the INDOT Safety Improvement Program. This list is available from the Planning Division. Each 3R project should be coordinated with proposed safety projects, since the implementation of projects in one area may influence priorities in another. A safety project and a 3R project should be accomplished at the same time as practical.

55-5.02 Obstruction-Free Zone

The obstruction-free zone is defined as the roadside area next to the travelway which should be free from hazards or obstructions. This is not the same as the clear zone, so these two terms are not interchangeable. Each obstacle within the obstruction-free-zone limits should be removed, made breakaway, or shielded with guardrail. The obstruction-free-zone widths shown below are minimums and should be extended where accident experience indicates that a wider zone would further enhance safety. The clear-zone width described in Section 49-2.0 should be provided, if practical. The designer should review Section 49-2.0 for additional information on clear zone. The following obstruction-free-zone requirements apply.

1. Arterial with Shoulders. Where the design speed is 50 mph or higher and the design-year AADT is greater than 1500, the minimum obstruction-free-zone width is 20 ft from the edge of the travelway, or from the edge of the travelway to the right-of-way line, whichever is
less. For all other situations, the minimum obstruction-free-zone width is 10 ft plus the minimum paved-shoulder width shown in Figure 55-3A, 55-3E, or 55-3F from the edge of the travelway, or from the edge of the travelway to the right-of-way line, whichever is less.

2. **Collector with Shoulders.** Where the design speed is 50 mph or higher and the design-year AADT is greater than 1500, the minimum obstruction-free-zone width is 10 ft plus the minimum paved-shoulder width shown in Figure 55-3B, 55-3C, or 55-3G from the edge of the travelway, or from the edge of the travelway to the right-of-way line, whichever is less. For all other situations, the minimum obstruction-free-zone width is 6 ft plus the minimum paved-shoulder width shown in Figure 55-3B, 55-3C, or 55-3G from the edge of the travelway, or from the edge of the travelway to the right-of-way line, whichever is less.

3. **Local Road or Street with Shoulders.** The minimum obstruction-free-zone width is 6 ft plus the usable-shoulder width shown in Figure 55-3D or 55-3H from the edge of the travelway, or from the edge of the travelway to the right-of-way line, whichever is less.

4. **Curbed Roadway.** Where the design speed is 45 mph or lower, and curbs are at least 6 in. in height, the minimum obstruction-free-zone width from the face of the curb should be 1.5 ft. However, where traffic-signal supports are present, the minimum obstruction-free-zone width should be 2.5 ft. Where the design speed is 50 mph or higher regardless of curb height, or curbs are less than 6 in. in height regardless of design speed, the minimum obstruction-free-zone width should be as defined in Item 1, 2, or 3 above.

5. **Appurtenance-Free Zone.** There should be a 1.5-ft appurtenance-free area from the front face of curb or from the edge of the travel lane if there is no curb. Where traffic-signal supports are present, a 2.5-ft clear width should be provided. The appurtenance-free zone is defined as an area in which nothing, including breakaway safety appurtenances, should protrude above the paved or earth surface (see Figure 55-5A, 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, the opening of vehicular doors, etc.

6. **On-Street Parking.**

   a. **Continuous 24-Hour Parking.** No obstruction-free zone is required where there is continuous 24-h parking. However, the appurtenance-free zone shown in Figure 55-5A should be provided from the front face of the curb or the edge of the parking lane if there is no curb.
b. Parking Lane Used as a Travel Lane. The obstruction-free zone should be
determined assuming the edge of the parking lane as the right edge of the farthest-
right travel lane.

55-5.03 Treatment of Obstruction

An obstruction or non-traversable hazard within the obstruction-free zone should be, in order of
preference, as follows:

1. removed or redesigned so that it can be safely traversed;
2. relocated outside of the obstruction-free 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.

55-5.03(01) Application

The following hazards should be eliminated or modified, according to the treatment hierarchy
described above, if they are within the obstruction-free zone:

1. Tree. A tree maturing to a diameter of 4 in. or greater should be removed from the
obstruction-free zone, unless shielded by a protective device required for other purposes. A
tree on a backslope may remain if it is unlikely to be impacted by an errant vehicle.

2. Obstruction. An obstruction such as a rough rock cut, boulder, headwall, foundation, etc.,
with projections that extend more than 4 in. above the ground line should be removed,
relocated, made breakaway, or shielded with guardrail as appropriate. A rough rock cut is
one that presents a potential vehicular snagging problem.

3. Sign or Light Support. Each signpost or light pole to remain within the obstruction-free
zone should be made breakaway. In an urban area where pedestrian traffic is prevalent, a
breakaway light support should not be used. However, such a support should, as a
minimum, be offset beyond the obstruction-free-zone width as described in Section 55-
5.02, desirably behind the sidewalk. In a rural area where pedestrian traffic is prevalent,
the use of a breakaway support will be considered by the field-review team. Section 49-3.06 provides additional information on the treatment of a sign or light support within the obstruction-free zone.

4. Traffic Signal. A traffic-signal support should be placed to provide the obstruction-free zone through the area where the traffic-signal supports are located. However, the following exceptions will apply.

a. Channelized Island. Installation of a signal support in a channelizing island should be avoided, if practical. However, if a signal support must be located in a channelizing island, a minimum clearance of 30 ft should be provided in a rural area from all travel lanes, including turn lanes, or in an urban where the design speed is 50 mph or higher. In an urban area where the island is bordered by a vertical curb and the design speed 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, Design Speed ≥ 50 mph or AADT > 1500. Where conflicts exist such that the placement of traffic-signal supports outside the obstruction-free zone is impractical (e.g., conflicts with buried or utility cables), the signal supports should be located at least 10 ft beyond the outside edge of the shoulder.

c. Non-Curbed Facility, Design Speed ≤ 45 mph or AADT ≤ 1500). Where conflicts exist such that the placement of traffic-signal supports outside the obstruction-free zone is impractical (e.g., conflicts with buried or utility cables), the signal supports should be located at least 6 ft beyond the outside edge of the shoulder.

5. Culvert. A culvert end is considered to be within the obstruction-free zone if the point at which the top of the culvert protrudes from the slope is within the obstruction-free zone. Section 55-5.03(02) provides additional information for the treatment of a drainage structure.

6. Transverse Slopes of Public Road Approach or Drive. Steep transverse slopes should be considered for flattening, if practical. Such slopes should desirably be 6:1 or flatter, not steeper than 4:1. Transverse slopes on a median crossover should be 10:1 or flatter.

7. Curbs. Curbs should be removed from a rural highway where the design speed is 50 mph or higher. The proper placement of traffic control devices must be considered in reviewing the removal of corner island curbs where such devices are located. This is not intended to address divisional, or channelizing, islands separating two-way traffic or a curb placed at the edge of a shoulder for drainage. For these situations, sloping curbs should be used.
Curbs of at least 4 in. in height should not be used in conjunction with guardrail. The front face of a curb used in conjunction with guardrail should desirably be behind the face of the rail. If this cannot be achieved, the front face of the curb may be located flush with the face of the rail.

8 **Utility Pole.** A utility pole within the obstruction-free zone which is not owned by INDOT or a local agency can constitute a significant hazard and should be removed or relocated. The utility company should be requested to relocate poles that are located in a high-vulnerability area such as a channelizing island, or where the accident history indicates there has been a utility-pole-accident problem. The field-review team, based on its judgment, will determine where such work is warranted.

9. **Mailbox Support.** Each new mailbox installation should be placed in accordance with the INDOT Standard Drawings, INDOT Standard Specifications, and Section 51-11.0.

10. **Non-Traversable Hazard.** A fill slopes steeper than 1:1 with a height greater than 2 ft within the obstruction-free zone should be flattened to the extent practical. If part of a drainage ditch appears within the obstruction-free zone, its cross section should be in accordance with the criteria described in Section 49-3.02.

11. **Drainage Ditch.** A ditch is considered inside the obstruction-free zone if the near side of the ditch bottom is within the obstruction-free zone.

   If a ditch is located inside the obstruction-free zone, the ditch should be traversable. See Section 49-3.02. If the ditch it is not traversable, a Level Two design exception is required. If a traversable ditch is not provided, a 4-ft width bottom should be provided for the ditch with the backslope as flat as practicable.

   If a ditch is located outside the obstruction-free zone, it can be made traversable. However, it is not mandatory to provide a traversable-ditch section. This can be accomplished but should only be pursued where the gentler section does not significantly affect the right-of-way needs. This should be determined during the field review, and can be accomplished as follows:

   a. a 4-ft flat-bottom ditch should be provided;

   b. a flat-bottom ditch of less than 4 ft width should be provided; or

   c. a V ditch should be provided.
The backslope should be designed to be as flat as practicable.

12. **Other Hazard.** The designer should review Section 49-3.0 to determine the appropriate treatment for other hazards not discussed above, such as a bridge pier or bridge-railing end.

### 55-5.03(02) Drainage Structure

A mainline cross culvert of 60 in. diameter or less, or a pipe-arch 83 in. x 57 in. or smaller, should not be extended to locate the inlet and outlet ends outside the obstruction-free zone. This practice can introduce undesirable embankment slope discontinuities. A structure which is terminated within the obstruction free zone should be treated as follows:

1. Standard metal culvert-end sections should be used within the obstruction-free zone with a circular culvert of 30 in. diameter or less, or with a pipe-arch culvert of 36 in. x 24 in. or less, either of which is skewed 10 deg or less from the perpendicular, towards the direction of approaching traffic.

2. Grated-box end sections should be used with a circular culvert of diameter of greater than 36 in. through 60 in., or with a pipe-arch culvert of 45 in. x 27 in. through 83 in. x 57 in.

3. Grated-box end sections should be used with a culvert which is skewed more than 10 deg from the perpendicular, towards the direction of approaching traffic.

4. If the end of a culvert of 66 in. diameter or larger is within the obstruction-free zone, guardrail should be provided. If the culvert end is outside the obstruction-free zone, the designer should use engineering judgment to determine if it is desirable to protect an errant motorist from the culvert end with guardrail. If there is inadequate height of cover to drive the guardrail posts, the treatment shown for guardrail over a low-fill culvert in Section 49-5.03 and the INDOT *Standard Drawings* should be used.

5. If the point at which the top of a box culvert or three-sided structure protrudes from the slope is within the obstruction-free zone, guardrail should be provided. Otherwise, Figure 55-5A(1), Clear Zone / Guardrail at Culvert, should be used to determine the appropriate treatment.

Each culvert of 12 in. or 15 in. diameter that is parallel to the mainline and inside the obstruction-free zone, or is within a median of 60 ft width or less, requires standard metal or concrete end sections. Each culvert of greater than 15 in. diameter that is parallel to the mainline and inside the obstruction-free zone, or is within a median of 60 ft width or less, requires grated-box end sections.
55-5.04 Roadside Barrier

Each existing safety appurtenance should be examined to determine if it is in accordance with the current safety performance and design criteria. This includes guardrail, median barrier, impact attenuator, sign support, luminaire support, or bridge railing. Substandard safety appurtenances should be upgraded to be in accordance with the current safety performance and design criteria. Chapter 49 and the INDOT Standard Drawings provide the Department’s criteria for the layout and design of safety appurtenances.

55-5.04(01) Existing Guardrail

An existing guardrail installation should be removed where such installation is not in accordance with the location warrants described in Section 49-4.0 or where the obstacle or hazard can be removed at a cost of less than guardrail upgrading plus estimated guardrail maintenance costs over the life of the installation. If existing guardrail is still warranted, it should be upgraded as follows:

1. **Guardrail Components.** Each guardrail and end treatment which is not in accordance with Section 49-4.0 and the INDOT Standard Drawings should be replaced or upgraded to the current criteria. However, existing W-beam guardrail with U-channel rubrail may be retained. An existing buried-end section may remain on a two-lane local-agency route if the design-year AADT is less than 1000.

2. **Transition.** Each substandard guardrail transition to a bridge pier or other obstruction should be upgraded or replaced to be in accordance with Section 49-4.0 and the INDOT Standard Drawings.

3. **Height.** Guardrail of less than 2'-3” height at the top of the rail element should be raised using adjustable blockouts, or reset or replaced as appropriate.

4. **Lateral Clearance.** Reduced post spacing should be provided where the distance between guardrail and an obstruction is less than the required deflection distance shown in Section 49-5.0.

5. **Gap.** Each gap of 200 ft or less between guardrail runs should be closed, if practical.

6. **Length of Need.** Each guardrail run’s length of need should be in accordance with Section 49-5.0. The obstruction-free-zone width shown in Section 55-5.02 should not be used as the clear-zone width in determining the length-of-need requirement. The clear-zone width for computing the length of need is shown in Section 49-2.01. The length of need may be
modified if deemed appropriate by the field-review team. See Figure 55-5B, Runout Length, $L_R$, (ft) for Restrictive Condition.

55-5.04(02) New Guardrail Installation [Rev. May 2013]

New guardrail should be installed as follows:

1. where it is not practical to eliminate an obstacle from the obstruction-free zone as defined in Section 55-5.03;

2. where the guardrail is judged to be less hazardous than the obstacle;

3. at each approach to a bridge railing; and

4. where in the opinion of the field-review team, there is an extreme hazard which obviously warrants guardrail.

Each new installation of guardrail should be in accordance with Chapter 49 and the INDOT Standard Drawings, except as follows.

1. **Length of Need.** The length of need may be modified by the field-review team if deemed absolutely necessary.

2. **Guardrail Offset.** The desirable guardrail offset is 2 ft from the effective usable-shoulder width, or the shy-line offset distance, whichever is larger. See Figure 49-5F for shy-line offset. In a restrictive situation, depending on functional classification, this distance may be 0 ft. The minimum guardrail offset distance is 4 ft from the edge of travelway.

3. **Post Embedment and Earth Backup.** The desirable distance from the face of guardrail to the shoulder break point is 3'-5" ft. This provides a 2-ft offset from the back of the guardrail post. In a restrictive situation, the offset from the back of the guardrail post may be 0 ft.

4. **End Treatment.** The type I end treatment may not be used on an INDOT route, or other facility which has a design year-traffic volume of 1000 AADT or greater. Section 49-5.04 provides additional information on end treatments which may be used on a high-volume, high-speed road.
5. **Length of Need for Restrictive Condition.** Where a restrictive condition warrants, Figure 55-5B, Runout Length, $L_R$ (ft), for Restrictive Condition, should be used

One example of a restrictive condition is the proximity of a drive to the end of a bridge, which cannot be relocated farther from the bridge.

If it is decided at the field check to shorten a guardrail run’s length of need, the field check minutes must document the decision.

### 55-6.0 BRIDGE

**55-6.01 General Requirements**

An existing bridge may remain in place if it satisfies, or is upgraded to satisfy, the structural and geometric requirements shown in the appropriate figure in the 55-3 series, and in Section 55-6.02. Upgrading a bridge to satisfy the criteria should only be undertaken if an engineering and economic analysis shows that the upgrading is cost effective. Some of the considerations for such an analysis include the following:

1. remaining service life;
2. sufficiency rating;
3. traffic volume;
4. clear-roadway width;
5. snow storage;
6. farm equipment clearances;
7. design speed; and
8. accident records.

If it is decided that a bridge should be replaced or have major reconstruction (e.g., replace superstructure, widen superstructure, or widen substructure), the design should be done in accordance with the appropriate AASHTO criteria and load-carrying capacity (see Chapter 60). The only exception is that the bridge-width criteria shown in Section 55-6.03 may be used if the most likely level of future (20 to 30 years) highway improvement on the approaches and adjacent road sections will be to 3R criteria (i.e., the road will not be reconstructed in the foreseeable future).

Reasons for determining the use of the width shown in Section 55-6.03 must be documented in the Preliminary Engineering Study for an INDOT-route project, or in the Safety and Design Report for a local-agency-route project. The width shown in Section 55-6.03 may also be used for a bridge which is part of a 3R project, an isolated bridge on existing alignment, or an isolated bridge where
the alignment has been changed. In the latter situation, the minor-roadway realignment may be constructed to 3R criteria as described in this chapter.

55-6.02 Bridge To Remain In Place

If an existing bridge is structurally sound and if is in accordance with the appropriate AASHTO design loading for structural capacity, it is unlikely to be economical to improve the geometrics of the bridge. If an existing bridge is not in accordance with the following, it should be evaluated for upgrading or replacing (see Section 55-6.01). The following will apply to an existing bridge.

1. **Width.** The width should be evaluated against the criteria shown in the appropriate figure in the 55-3 series.

2. **Structural Capacity.** The structural capacity should be evaluated against the criteria shown in the appropriate figure in the 55-3 series.

3. **Vertical Clearance.** An existing structure should provide at least a 14.0-ft vertical clearance. If this vertical clearance is not available, consideration should be given to increasing the vertical clearance either as part of the 3R project or as a separate project. Modifications should desirably provide for a clearance of 14.5 ft. If it is necessary to retain a vertical clearance of less than 14.0 ft, a design exception request must be processed in accordance with Section 40-8.0. Low-clearance signage is required for a vertical clearance of less than 14.5 ft.

4. **Bridge Railing.** Only existing bridge railing that has been proven to be acceptable through crash testing or that satisfies the structural and geometric requirements of the AASHTO LRFD Bridge Design Specifications may be retained. Each new bridge-railing installation must be in accordance with Section 61-6.01. Consideration should be given to widening the bridge at the same time the railing is replaced to achieve the full approach travelway and shoulder width.

A design exception to this criterion will only be considered if all of the conditions are satisfied as follows:

a. the project is a rehabilitation project on a non-NHS route;

b. the existing bridge railing and approach guardrail are considered to be satisfactory;

c. the accident history does not indicate that there may be a problem;
d. the design year AADT is less than 400; and

e. the design speed is 30 mph or lower.

5. **Narrow Bridge.** Each bridge which is narrower than the approach roadway width, and is not to be widened, should be signed and pavement-marked as shown on the INDOT Standard Drawings. NCHRP 203 Safety at Narrow Bridge Sites provides criteria specifically for a narrow bridge, e.g., pavement markings.

### 55-6.03 Bridge Requiring Replacement or Major Reconstruction

The bridge clear-roadway width shown in the appropriate figure in the 55-3 series is intended to be applied only to a bridge where it has been determined that the 3R criteria is the most probable level of future (20 to 30 years) highway improvement on the approaches and adjacent roadway sections. If the expected improvement will be reconstruction, the width shown in the appropriate figure in the 55-3 series should be used. The 3R bridge work may include rehabilitation using structurally-sound elements of an existing bridge, complete bridge replacement on existing alignment, or a replacement bridge on a short relocation. This width is a minimum, and a greater width should be used if deemed appropriate.

The minimum clear-roadway width is the sum of the lane widths and useable shoulder widths (or curb-offset widths) shown in the appropriate figure in the 55-3 series, plus the bridge railing offset distance from Figure 402-6H. The intent is to carry the roadway cross section across the bridge. The minimum clear-roadway width should be 30 ft on a rural INDOT route. The width must be at least 30 ft if bridge-deck rehabilitation is to be done on one-half the width at a time. This can eliminate the need for a detour or runaround, or the use of a local road to re-route traffic. If it is determined that it is practical to close the bridge and detour traffic, the 30-ft minimum may not be necessary.

The use of the road by agricultural equipment may necessitate the use of a clear-roadway width greater than the minimum prescribed herein. The need for a greater width to accommodate such equipment will be determined for each project. Approach guardrail should be offset to the same position as the bridge railing from the edge of the traveled way, if a clear-roadway width greater than that of the approach roadway (traveled way plus shoulders) is used.

Each bridge must be designed to comply with the AASHTO load-carrying capacity requirement shown in the appropriate figure in the 55-3 series. Each new bridge-railing installation must be in accordance with Section 404-4.0. The waterway opening will be determined in accordance with the applicable environmental-permit requirements.
55-7.0 MISCELLANEOUS DESIGN ELEMENTS

55-7.01 Traffic-Control Devices

All signs, traffic signals, and pavement markings on the mainline and intersections, and related traffic-control devices on public road approaches must be in accordance with Part VII and the MUTCD. Center-line and edge-line pavement markings, no-passing zone warning signs, and regulatory signs are required. It may be necessary to extend pavement markings and place related signs beyond the project limits to end them at a logical terminus (e.g., major intersection, end of a no-passing zone). Center lines and edge lines need not be installed where they are not warranted, based on the opinion of the field-review team. For example, pavement markings would not be warranted on a bridge-replacement project on a road that does not have pavement markings.

55-7.02 Railroad Crossing Warning Devices and Surface

The adequacy of existing warning devices and crossing surface should be investigated if the 3R project includes an at-grade railroad crossing within the project limits. A railroad grade-crossing surface should provide for a reasonably smooth ride and should have a width equal to at least the approach traveled way plus shoulders plus 1 ft on each side. A railroad crossing which does not satisfy the above surface requirements should be upgraded concurrent with the 3R work. If an active warning-device installation or upgrading is determined to be necessary, it should also be done concurrent with the 3R project. For more information on upgrading an at-grade railroad crossing, see Chapters 11 and 47.

55-7.03 Trimming of Trees and Brush

Trees and brush should be trimmed, as necessary, to obtain the required stopping, intersection, or railroad-crossing sight distance and signage visibility.

55-7.04 Encroachment

Each encroachment within the right of way should be treated in accordance with Section 86-2.0.
55-8.0 ACCIDENT DATA ANALYSIS

A primary measure of the safety of an existing highway is its accident history. Once a highway location has been proposed, accident data should be collected and analyzed to determine the relative safety of the facility and to identify and describe the accident characteristics or patterns that have occurred. Safety enhancements to alleviate safety deficiencies can be more-readily identified from this analysis, and the extent of minimum safety enhancement can be determined.

55-8.01 Accident-Analysis Procedures

55-8.01(01) Responsibilities

In conducting an accident analysis, the duties to be performed are as follows:

1. be prepared to spend sufficient time conducting the accident study;

2. study individual accident reports;

3. check project termini, often at some logical point such as an intersection, to ensure that accident information is considered just beyond the project termini;

4. relate accident data to field conditions, preferably if there are only a limited number of accidents reported. Review the data in the field or on the videolog; and

5. discuss the project with maintenance personnel. Many single-vehicle or non-injurious accidents are unreported and yet are strong indicators of potentially hazardous situations.

55-8.01(02) Accident Summaries

Accident analysis study procedures involve determining the significance of the accident history and the development of summaries of the accident characteristics within the 3R project termini. The project’s accident summaries are used to detect abnormal accident trends or patterns and to distinguish between correctable and non-correctable accident experience. Analysis of these summaries is needed to identify probable safety deficiencies of the existing facility.

In conducting the accident analysis, the following should be considered.
1. **Time Period.** The required time period for the collection of the accident history is three years. In selecting the period, the accident data should represent reasonably current information, as related factors such as traffic volume, pavement condition, or other site-related data can vary with time. Likewise, the past changes in the character of the facility (e.g., physical changes, roadside development) should be accounted for when evaluating the accident activity.

2. **Vehicle Directions.** The accident data should be examined to determine the directions the vehicles were traveling.

3. **Location.** Accident data should be examined with respect to location. Accidents occurring within an intersection area should be separated from those occurring outside the area of influence of the intersection. Similar accident types occurring in differing situations should be separated. For example, left-turn accidents into a drive should not be included with left-turn accidents at an intersection.

4. **Accident Rate.** The accident data should be examined to determine the number of accidents and the accident rates within the project termini. Limited accident data is likely on a rural 2-lane highway with a low to moderate traffic volume. The limited amount of such data can make traditional methods of analysis difficult. Accident rates generated from a small sample can be misleading as they can be significantly influenced by small variances.

5. **Summary Form.** The accident data should be summarized by type and severity. Figure 55-8A, Accident Analysis Form, provides a typical accident summary form that may be used to analyze accidents. An editable version of this form may also be found on the Department’s website at [www.in.gov/dot/div/contracts/design/dmforms/](http://www.in.gov/dot/div/contracts/design/dmforms/). Figure 55-8B, Accident Analysis Form Codes, and Figure 55-8C, Collision Diagram Codes, provide the codes which are used in conjunction with Figure 55-8A.

6. **Accident Analysis.** Once the accident data has been compiled, the data should be reviewed to identify accident patterns and determine possible causes for the accident patterns. The severity patterns should be examined to determine if a particular roadway or roadside feature may have contributed to the overall severity of the accidents that have occurred. Section 55-8.02 provides additional information on probable accident causes and possible safety enhancements.

7. **Contributing Factors.** The Contributing Circumstances portion of the accident report should be summarized. This identifies possible accident causes noted by the investigating police officer at the scene of the accident. Contributing circumstances are categorized by human
(driver), environmental, or vehicle-related factors. The contributing-circumstances information is used to verify, add, or delete possible causes developed by the accident-summary-by-type procedure. The contributing-circumstances information can be used to separate correctable and non-correctable accidents. In separating the accidents by these classifications, consideration should be made to ensure that the accidents are indeed non-correctable. Figure 55-8D lists the contributing circumstances found on most accident reports, and if they are correctable or non-correctable through highway improvements.

8. **Environmental Factors.** Accidents should be summarized by environmental conditions. This procedure identifies possible causes of safety deficiencies related to the existing condition of the roadway environment at the time of the accident. Typical classifications used in the analysis include lighting conditions and roadway surface condition. The summary is compared to average or expected values for similar locations or areas to determine whether the occurrence of a specific environmental characteristic is greater or less than the expected value at the location. For example, a higher-than-expected number of wet-surface accidents may be an indication of slippery pavement.

**55-8.02 Probable Causes and Safety Enhancements**

Probable accident causes should be defined once the accident patterns are identified. Field conditions, as determined by an on-site or videolog review, or from information on the police accident report or computerized accident form, should be used to refine the list of possible causes to the most probable. The identified probable causes can then be used as a basis for selecting appropriate safety enhancements to alleviate the safety deficiency. Figure 55-8E, Accident Analysis, provides a list of probable accident causes and possible safety enhancements. This list is not all-inclusive. However, it does provide a general list of possible accident causes as a function of accident patterns and appropriate safety enhancements.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2-Lane</th>
<th>Multi-Lane</th>
</tr>
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<tr>
<td><strong>Design Controls</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design-Year AADT</td>
<td>40-2.01</td>
<td>&lt; 400 400 ≤ AADT 3000 ≤ AADT ≥ 5000</td>
<td>Undivided</td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>55-4.01</td>
<td>20 Years (1)</td>
<td>20 Years (1)</td>
</tr>
<tr>
<td><em>Design Speed, mph (2)</em></td>
<td>55-4.01</td>
<td>Posted Speed Limit</td>
<td>Posted Speed Limit</td>
</tr>
<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>Partial Control / None</td>
<td>Partial Control / None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Desirable: B; Minimum: D</td>
<td>Desirable: B; Minimum: D</td>
</tr>
<tr>
<td><strong>Cross-Section Elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Travel Lane</td>
<td>*Width</td>
<td>55-4.05</td>
<td>12 ft 12 ft 12 ft 12 ft</td>
</tr>
<tr>
<td></td>
<td>Typical Surface Type (3)</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>Shoulder (4)</td>
<td>*Width Usable</td>
<td>55-4.05</td>
<td>D: 6 ft M: 2 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: 8 ft M: 3 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: 8 ft M: 6 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: 11 ft M: 8 ft</td>
</tr>
<tr>
<td></td>
<td>*Width Paved</td>
<td>55-4.05</td>
<td>D: 4 ft M: 0 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: 6 ft M: 2 ft</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>D: 6 ft M: 2 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>D: 10 ft M: 2 ft</td>
</tr>
<tr>
<td>Cross-Slopes</td>
<td>*Travel Lane (5)</td>
<td>55-4.05</td>
<td>2%</td>
</tr>
<tr>
<td></td>
<td>Shoulder (6)</td>
<td>55-4.05</td>
<td>Paved Width ≤ 4 ft 2%; Paved Width &gt; 4 ft 4% Asphalt / Concrete: 6% Sealed Aggregate</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Paved Width ≤ 4 ft 2%; Paved Width &gt; 4 ft 4% Asphalt / Concrete: 6% Sealed Aggregate</td>
</tr>
<tr>
<td>Auxiliary Lane</td>
<td>Lane Width</td>
<td>55-4.05</td>
<td>Desirable: 12 ft; Minimum: 11 ft</td>
</tr>
<tr>
<td></td>
<td>Shoulder Width</td>
<td>55-4.05</td>
<td>Des: Same as Next to Travel Lane; Min: 2 ft</td>
</tr>
<tr>
<td>Median Width</td>
<td>55-4.05</td>
<td>N/A</td>
<td>0.0 ft, Existing</td>
</tr>
<tr>
<td>Obstruction-Free-Zone Width</td>
<td>55-5.02</td>
<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
</tr>
<tr>
<td>Side Slopes</td>
<td>Cut Foreslope</td>
<td>55-4.05</td>
<td>2:1 or Flatter (7)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(7)</td>
</tr>
<tr>
<td></td>
<td>Ditch Width</td>
<td></td>
<td>2:1 or Flatter (7)</td>
</tr>
<tr>
<td></td>
<td>Backslope</td>
<td></td>
<td>2:1 or Flatter (7)</td>
</tr>
<tr>
<td>Median Slopes</td>
<td>55-4.05</td>
<td>N/A</td>
<td>Desirable: 8:1; Maximum: 4:1</td>
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<tr>
<td><strong>Bridges</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>New or Reconstructed Bridge</td>
<td>*Structural Capacity</td>
<td>Ch. 403</td>
<td>HL-93 (8)</td>
</tr>
<tr>
<td></td>
<td>*Clear-Roadway Width (9)</td>
<td>55-6.03</td>
<td>Full Paved Approach Width</td>
</tr>
<tr>
<td>Existing Bridge to Remain in Place</td>
<td>*Structural Capacity</td>
<td>Ch. 72</td>
<td>HS-20</td>
</tr>
<tr>
<td></td>
<td>*Clear-Roadway Width</td>
<td>55-6.02</td>
<td>Travelway Plus 2 ft on Each Side</td>
</tr>
<tr>
<td><strong>Vertical Clearance, Arterial Under (10)</strong></td>
<td>New or Replaced Overpassing Bridge</td>
<td>55-6.0</td>
<td>16.5 ft</td>
</tr>
<tr>
<td></td>
<td>Existing Overpassing Bridge (11)</td>
<td></td>
<td>14.0 ft</td>
</tr>
<tr>
<td></td>
<td>Sign Truss / Pedestrian Bridges</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Clearance, Arterial Over Railroad (12)</td>
<td>402-6.01</td>
<td>23.0 ft</td>
<td></td>
</tr>
</tbody>
</table>

D or Des: Desirable; M or Min: Minimum

* Level One controlling criterion, see page 2 of 4.

GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIAL, 3R PROJECT

Figure 55-3A (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
</tr>
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<tbody>
<tr>
<td>Design Speed</td>
<td>50 mph</td>
</tr>
<tr>
<td>*Stopping Sight Distance, Desirable</td>
<td>425 ft</td>
</tr>
<tr>
<td>*Superelevation Rate</td>
<td>See Section 55-4.03</td>
</tr>
<tr>
<td>Decision Sight Distance Path / Direction Change</td>
<td>750 ft</td>
</tr>
<tr>
<td>Stop Maneuver</td>
<td>465 ft</td>
</tr>
<tr>
<td>Passing Sight Distance</td>
<td>Existing</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (14)</td>
<td>55-4.06 P: 630 ft; SUT: 780 ft</td>
</tr>
<tr>
<td>*Minimum Radius</td>
<td>55-4.03 See Section 55-4.03</td>
</tr>
<tr>
<td>*Horizontal Sight Distance</td>
<td>See Section 55-4.03</td>
</tr>
<tr>
<td>*Vertical Curvature, K-value</td>
<td>See Section 55-4.04</td>
</tr>
<tr>
<td>*Maximum Grade (13)</td>
<td>55-4.04 5%</td>
</tr>
<tr>
<td>*Minimum Grade (13)</td>
<td>4%</td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03 Desirable: 0.5%; Minimum 0.0%</td>
</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.
(1) Design Forecast Period. For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(2) Design Speed. The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit, 60 mph, on a non-posted multilane divided highway, or 55 mph on a non-posted two-lane highway.

(3) Surface Type. The pavement-type selection will be determined by the Pavement Engineering Division or by the local jurisdiction.

(4) Shoulder. The following will apply:
   a. On an INDOT facility, the shoulder should be paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. In a restrictive situation, the guardrail offset may be 0 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. If guardrail is present, the minimum offset from E.T.L. to the front face of guardrail should desirably be equal to the shy-line distance, but should not be less than 4 ft. See Section 49-4.0 for shy-line offsets.
   c. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(5) Cross Slope, Travel Lane. Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(6) Cross Slope, Shoulder. Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

(7) Side Slopes. Section 55-4.05 provides additional information for side-slope criteria.

(8) Structural Capacity, New or Reconstructed Bridge. The following will apply:
   a. Each State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. Each bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.
(9) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. On a State highway, the minimum clear-roadway width should be 30 ft. Otherwise, the bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled-way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(10) **Vertical Clearance, Arterial Under.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulders.

(11) **Vertical Clearance, Existing Bridge.** See Section 55-6.02 for additional information on minimum allowable vertical clearance.

(12) **Vertical Clearance, Arterial Over Railroad.** See Section 402-6.01(03) for additional information on railroad clearance under a highway.

(13) **Maximum Grade.** A downgrade that is 1% steeper may be used for a one-way roadway.

(14) **Intersection Sight Distance.** For left turn onto a 2-lane road. P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2-Lane AADT</th>
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</thead>
<tbody>
<tr>
<td>Design Controls</td>
<td></td>
<td>&lt; 400</td>
</tr>
<tr>
<td>Design-Year AADT</td>
<td>40-2.01</td>
<td>400 ≤ AADT &lt; 1000</td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>55-4.01</td>
<td>20 Years (1)</td>
</tr>
<tr>
<td>*Design Speed (mph)</td>
<td>55-4.01</td>
<td>Posted Speed Limit</td>
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<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Desirable: B; Minimum: D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cross-Section Elements</th>
<th>Manual Section</th>
<th>2-Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Travel Lane</td>
<td>*Width</td>
<td>55-4.05</td>
</tr>
<tr>
<td>Typical Surface Type</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>Shoulder (5)</td>
<td>*Width Usable</td>
<td>55-4.05</td>
</tr>
<tr>
<td>*Width Paved</td>
<td>55-4.05</td>
<td>Des: 2 ft</td>
</tr>
<tr>
<td>Typical Surface Type</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete / Sealed Aggregate</td>
</tr>
<tr>
<td>Cross Slope</td>
<td>*Travel Lane</td>
<td>55-4.05</td>
</tr>
<tr>
<td>Shoulder (7)</td>
<td>55-4.05</td>
<td>Paved Width ≤ 4 ft: 2% - 3%; Paved Width &gt; 4 ft: 4%-6% Asphalt / Concrete; 6% Sealed Aggregate</td>
</tr>
<tr>
<td>Auxiliary Lane</td>
<td>Lane Width</td>
<td>55-4.05</td>
</tr>
<tr>
<td>Shoulder Width</td>
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</tr>
<tr>
<td>Obstruction-Free-Zone Width</td>
<td>55-5.02</td>
<td>See Section 55-5.02</td>
</tr>
<tr>
<td>Side Slopes</td>
<td>Cut</td>
<td>Foreslope</td>
</tr>
<tr>
<td></td>
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<td>Ditch Width</td>
</tr>
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<td></td>
<td></td>
<td>Backslope</td>
</tr>
<tr>
<td></td>
<td>Fill</td>
<td>55-4.05</td>
</tr>
<tr>
<td>Bridges</td>
<td>*Structural Capacity</td>
<td>55-6.02</td>
</tr>
<tr>
<td>New or Reconstructed Bridge</td>
<td>Ch. 403</td>
<td>HL-93</td>
</tr>
<tr>
<td>Existing Bridge to Remain in Place</td>
<td>Ch. 72</td>
<td>HS-15</td>
</tr>
<tr>
<td>*Clear-Roadway Width (10)</td>
<td>55-6.03</td>
<td>Full Paved Approach Width</td>
</tr>
<tr>
<td>*Structural Capacity</td>
<td>Ch. 72</td>
<td>HS-15</td>
</tr>
<tr>
<td>*Clear-Roadway Width (11)</td>
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<td>22 ft</td>
</tr>
<tr>
<td>New or Replaced Overpassing Bridge</td>
<td>55-6.0</td>
<td>14.5 ft</td>
</tr>
<tr>
<td>Existing Overpassing Bridge</td>
<td>Ch. 402-6.01</td>
<td>23.0 ft</td>
</tr>
</tbody>
</table>

Des: Desirable; Min: Minimum.
* Level One controlling criterion, see page 2 of 4.

GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, STATE ROUTE, 3R PROJECT
Figure 55-3B
(Page 1 of 4)
### Design Element

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<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2-Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Speed</td>
<td>---</td>
<td>35 mph</td>
</tr>
<tr>
<td>*Stopping Sight Distance, Desirable</td>
<td>55-4.02</td>
<td>45 mph</td>
</tr>
<tr>
<td>Speed / Path / Direction Change</td>
<td>525 ft</td>
<td>50 mph</td>
</tr>
<tr>
<td>Stop Maneuver</td>
<td>275 ft</td>
<td>55 mph</td>
</tr>
<tr>
<td>Speed</td>
<td>570 ft</td>
<td>60 mph</td>
</tr>
<tr>
<td>Decision Sight Distance</td>
<td>42-2.0</td>
<td>35 mph</td>
</tr>
<tr>
<td>Passing Sight Distance</td>
<td>42-3.0</td>
<td>45 mph</td>
</tr>
<tr>
<td>Stop Maneuver</td>
<td>275 ft</td>
<td>50 mph</td>
</tr>
<tr>
<td>Speed</td>
<td>395 ft</td>
<td>55 mph</td>
</tr>
<tr>
<td>Decision Sight Distance</td>
<td>465 ft</td>
<td>60 mph</td>
</tr>
<tr>
<td>Intersection Sight Distance, -3% to +3% (16)</td>
<td>55-4.06</td>
<td>35 mph</td>
</tr>
<tr>
<td>Minimum Radius</td>
<td>55-4.03</td>
<td>45 ft</td>
</tr>
<tr>
<td>Superelevation Rate</td>
<td>55-4.03</td>
<td>50 ft</td>
</tr>
<tr>
<td>Horizontal Sight Distance</td>
<td>55-4.03</td>
<td>55 mph</td>
</tr>
<tr>
<td>Vertical Curvature, K-value</td>
<td>55-4.04</td>
<td>60 mph</td>
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<tr>
<td>Level</td>
<td>9%</td>
<td>7.5%</td>
</tr>
<tr>
<td>Rolling</td>
<td>8%</td>
<td>7%</td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>8%</td>
</tr>
<tr>
<td>Maximum Grade (15)</td>
<td>55-4.04</td>
<td>6%</td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>8%</td>
</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO's *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.

These criteria apply to each project regardless of funding source.
(1) **Design Forecast Period.** For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(2) **Design Speed.** The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit, 55 mph, on a non-posted highway.

(3) **Travel Lane, Width.** A minimum 11 ft travel lane may be used where truck volume is less than 200 trucks per day.

(4) **Surface Type.** The pavement type selection will be determined by the Office of Pavement Engineering.

(5) **Shoulder.** The following will apply:
   a. The shoulder should be paved to the front face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. In a restrictive situation, the guardrail offset may be 1 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. If guardrail is present, the minimum offset from E.T.L. to the front face of guardrail should desirably be equal to the shy-line distance, but not less than 4 ft. See Section 49-4.0 for shy-line offsets.
   c. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(7) **Cross Slope, Shoulder.** Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

(8) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.

(9) **Structural Capacity, New or Reconstructed Bridge.** The following will apply:
   a. Each State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road Loading.
   b. Each bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.
(10) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. On a State highway, the minimum clear-roadway width should be 30 ft. Otherwise, the bridge clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(11) **Width, Existing Bridge to Remain in Place.** Clear width will be at least equal to the approach traveled way width or the value, whichever is greater.

(12) **Vertical Clearance, Collector Under.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulders.

(13) **Vertical Clearance, Existing Bridge.** See Section 55-6.02 for additional information on minimum allowable vertical clearance.

(14) **Vertical Clearance, Collector Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(15) **Maximum Grade.** For a grade less than 500 ft in length (PVT to PVC), the maximum grade may be up to 2% steeper than the value. For a road with AADT < 400, the maximum grade may also be 2% steeper.

(16) **Intersection Sight Distance.** For left turn onto a 2-lane road, P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>2-Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Controls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Year AADT</td>
<td>40-2.01</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>400 ≤ AADT &lt; 1000</td>
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<td></td>
<td></td>
<td>1000 ≤ AADT &lt; 3000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3000 ≤ AADT &lt; 5000</td>
</tr>
<tr>
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<td>Paved Width &gt; 4 ft 4%-6% Asphalt; 6%-8% Aggregate; 8% Earth</td>
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<td>2:1 or Flatter (8)</td>
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<td>Foreslope</td>
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<td>Fill</td>
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<td>2:1 or Flatter (8)</td>
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<td>HL-93</td>
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<td>Ch. 72</td>
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<td>22 ft</td>
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<td>28 ft</td>
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<tr>
<td>*Vertical Clearance, Collector Under</td>
<td>55-6.0</td>
<td>New or Replaced Overpassing Bridge (12)</td>
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<tr>
<td></td>
<td>55-6.0</td>
<td>14.5 ft</td>
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<td>Existing Overpassing Bridge</td>
</tr>
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<td></td>
<td>55-6.0</td>
<td>14.0 ft</td>
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<td>Ch. 402-6.01</td>
<td>23.0 ft</td>
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Des: Desirable; Min: Minimum.

* Level One controlling criterion, see page 2 of 4

**GEOMETRIC DESIGN CRITERIA FOR RURAL COLLECTOR, LOCAL-AGENCY ROUTE, 3R PROJECT**

Figure 55-3C (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>30 mph</th>
<th>35 mph</th>
<th>45 mph</th>
<th>50 mph</th>
<th>55 mph</th>
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<tr>
<td>Design Speed</td>
<td>---</td>
<td>30 mph</td>
<td>35 mph</td>
<td>45 mph</td>
<td>50 mph</td>
<td>55 mph</td>
</tr>
<tr>
<td>*Stopping Sight Distance, Desirable</td>
<td>55-4.02</td>
<td>200 ft</td>
<td>250 ft</td>
<td>360 ft</td>
<td>425 ft</td>
<td>495 ft</td>
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<tr>
<td>Decision Sight Distance</td>
<td>42-2.0</td>
<td>450 ft</td>
<td>525 ft</td>
<td>675 ft</td>
<td>750 ft</td>
<td>865 ft</td>
</tr>
<tr>
<td>Speed / Path / Direction Change Stop Maneuver</td>
<td>55-4.06</td>
<td>220 ft</td>
<td>275 ft</td>
<td>395 ft</td>
<td>460 ft</td>
<td>535 ft</td>
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<td>Passing Sight Distance</td>
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<td>Existing</td>
<td>Existing</td>
<td>Existing</td>
<td>Existing</td>
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<td>Intersection Sight Distance, -3% to +3% (15)</td>
<td>55-4.06</td>
<td>P: 330 ft</td>
<td>P: 390 ft</td>
<td>P: 500 ft</td>
<td>P: 630 ft</td>
<td>P: 730 ft</td>
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<td>*Superelevation Rate</td>
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<td>See Section 55-4.03</td>
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<td></td>
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<td>*Horizontal Sight Distance</td>
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<td>*Vertical Curvature, K-value</td>
<td>55-4.04</td>
<td>Crest: 9%</td>
<td>Crest: 9%</td>
<td>Crest: 8%</td>
<td>Crest: 8%</td>
<td>Crest: 7%</td>
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<tr>
<td>Sag</td>
<td>See Section 55-4.04</td>
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<tr>
<td>*Maximum Grade (14)</td>
<td>55-4.04</td>
<td>Level: 11%</td>
<td>Level: 10%</td>
<td>Level: 9%</td>
<td>Level: 9%</td>
<td>Level: 8%</td>
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<td>44-1.03</td>
<td>Desirable: 0.5%; Minimum: 0.0%</td>
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</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. See Section 40-8.0.
(1) **Applicability.** This figure is applicable only to a federal-aid funded project.

(2) **Design Forecast Period.** For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(3) **Design Speed.** The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit, 55 mph, on a non-posted highway.

(4) **Travel Lane, Width.** An 11 ft travel lane width should be used where truck volume exceeds 200 trucks per day. In addition, the following will apply:
   a. Where $V \geq 50$ mph, the minimum width is 10 ft.
   b. Where $V \geq 50$ mph, the minimum width is 11 ft.
   c. Where $V \geq 50$ mph, the minimum width is 12 ft.

(5) **Shoulder Width.** The following will apply:
   a. The desirable guardrail offset is 2 ft from the effective usable-shoulder width. See Section 49-4.0 for more information.
   b. If guardrail is present, the minimum offset from the E.T.L. to face of guardrail should desirably be equal to the shy-line offset distance, but not less than 4 ft (see Section 49-4.0 for shy-line offsets).
   c. Usable-shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(7) **Cross Slope, Shoulder.** Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

(8) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.
(9) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(10) **Structural Capacity, Existing Bridge to Remain in Place.** If the AADT ≤ 50, an HS-10 loading is acceptable.

(11) **Width, Existing Bridge to Remain in Place.** Clear-roadway width should be at least equal to the approach traveled way width or the value, whichever is greater. For a bridge of more than 100 ft in length, the value does not apply. The acceptability of such a bridge will be assessed individually.

(12) **Vertical Clearance, Collector Under.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulders.

(13) **Vertical Clearance, Collector Over Railroad.** See Chapter 402-6.01(3) for additional information on railroad clearance under a highway.

(14) **Maximum Grade.** For a grade of less than 500 ft in length (PVT to PVC), the maximum grade may be 2% steeper than the value. For a road with AADT < 400, the maximum grade may also be 2% steeper.

(15) **Intersection Sight Distance.** For left turn onto a 2-lane road, P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
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<th>Design Element</th>
<th>Manual Section</th>
<th>2-Lane</th>
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<td>Cross-Section Elements</td>
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<td>Des: 8 ft; Min: 6 ft</td>
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<td>Lane Width</td>
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<td>Shoulder Width</td>
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<td>Ditch Width</td>
<td>(8)</td>
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<td>Fill</td>
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<td>28 ft</td>
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<td>New or Replaced Overpassing Bridge</td>
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Des: Desirable; Min: Minimum.

* Level One controlling criterion, see page 2 of 4
<table>
<thead>
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<th>Manual Section</th>
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<td>42-2.0</td>
<td>450 ft</td>
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<td>42-3.0</td>
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</tr>
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<td><strong>Intersection Sight Distance, -3% to +3% (14)</strong></td>
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<td>S: 330 ft</td>
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<tr>
<td><strong>Minimum Radius</strong></td>
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<td>See Section 55-4.03</td>
</tr>
<tr>
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<td>See Section 55-4.03</td>
</tr>
<tr>
<td><strong>Vertical Curvature, K-value</strong></td>
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<td>See Section 55-4.04</td>
</tr>
<tr>
<td><strong>Maximum Grade</strong></td>
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<td>10%</td>
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<td><strong>Minimum Grade</strong></td>
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</tr>
</tbody>
</table>

* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.
(1) **Applicability.** This figure is applicable only to a federal-aid funded project.

(2) **Design Forecast Period.** For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(3) **Design Speed.** The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit, 55 mph, on a non-posted highway.

(4) **Travel Lane, Width.** An 11 ft travel lane should be used where truck volume exceeds 200 trucks per day. In addition, the following will apply:
   a. Where $V \geq 50$ mph, the minimum width is 10 ft.
   b. Where $V \geq 50$ mph, the minimum width is 11 ft.
   c. Where $V \geq 50$ mph, the minimum width is 12 ft.

(5) **Shoulder Width.** The following will apply:
   a. The desirable guardrail offset is 2 ft from the usable-shoulder width. In a restrictive situation, the guardrail offset may be 1 ft from the usable-shoulder width. See Section 49-5.0 for more information.
   b. If guardrail is present, the minimum offset from E.T.L. to face of guardrail should desirably be equal to the shy-line offset distance, but not less than 4 ft (see Section 49-5.0 for shy-line offsets).
   c. Usable shoulder width is defined as the distance from the edge of the travel lane to the shoulder break point.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(7) **Cross Slope, Shoulder.** Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

(8) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.
(9) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. Where shoulders are paved, it is desirable to provide the full roadway width across the bridge. Otherwise, the clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(10) **Structural Capacity, Existing Bridge to Remain in Place.** If the AADT \( \leq 50 \), an HS-10 loading is acceptable.

(11) **Width, Existing Bridge to Remain in Place.** A minimum clear-roadway width that is 2 ft narrower than that shown may be used on a road with few trucks. The clear-roadway width should be at least the same width as the approach travelway. For a one-lane bridge, the width may be 18 ft. For a bridge of more than 100 ft in length, the value does not apply. The acceptability of such a bridge will be assessed individually.

(12) **Vertical Clearance, Local Under.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulders.

(13) **Vertical Clearance, Local Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(14) **Intersection Sight Distance.** For left turn onto a 2 lane road, \( P \) = Passenger car; \( SUT \) = single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Built-Up</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Controls</td>
<td>55-4.01</td>
<td>20 Years (1)</td>
<td>20 Years (1)</td>
<td>20 Years (1)</td>
</tr>
<tr>
<td>*Design Speed, mph(2)</td>
<td>55-4.01</td>
<td>Posted Speed Limit</td>
<td>Posted Speed Limit</td>
<td>Posted Speed Limit</td>
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<tr>
<td>Access Control</td>
<td>40-5.00</td>
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<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.00</td>
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<td>Des: C; Min: D</td>
<td>Des: C; Min: D</td>
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<td>On-Street Parking</td>
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### Travel Lane

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<th>*Width (4)</th>
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<th>Ch. 52</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
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</thead>
<tbody>
<tr>
<td>Curbed: Des: 12 ft; Min: 11 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uncurbed: Des: 12 ft; Min: 11 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
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**Cross Section Elements**

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<thead>
<tr>
<th>Shoulder</th>
<th>*Paved Width (7)</th>
<th>Curb Offset (6)</th>
<th>Ch. 304</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curbed: Des: 5 ft; Min: 4 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uncurbed: Shld. Width +4 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
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<table>
<thead>
<tr>
<th>Shoulder</th>
<th>*Travel Lane (8)</th>
<th>Typical Surface Type (5)</th>
<th>Ch. 304</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right: 6 ft; Left: 3 ft</td>
<td></td>
<td></td>
<td>Des: 12 ft; Min: 11 ft</td>
<td>Des: 12 ft; Min: 10 ft</td>
<td>Des: 12 ft; Min: 10 ft</td>
<td></td>
</tr>
<tr>
<td>Paved Width ≤ 4 ft: 2%-3%; Paved Width &gt; 4 ft: 4%-6%</td>
<td></td>
<td></td>
<td></td>
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<table>
<thead>
<tr>
<th>Shoulder</th>
<th>Cross Slope</th>
<th>Typical Surface Type (5)</th>
<th>Ch. 304</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
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</thead>
<tbody>
<tr>
<td>Right: 4% - 6%; Left: 2% - 3%</td>
<td></td>
<td></td>
<td>Des: 12 ft; Min: 11 ft</td>
<td>Des: 12 ft; Min: 10 ft</td>
<td>Des: 12 ft; Min: 10 ft</td>
<td></td>
</tr>
<tr>
<td>Paved Width ≤ 4 ft: 2%-3%; Paved Width &gt; 4 ft: 4%-6%</td>
<td></td>
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<table>
<thead>
<tr>
<th>Shoulder</th>
<th>Auxiliary Lane</th>
<th>Typical Surface Type (5)</th>
<th>Ch. 304</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
<th>Asphalt / Concrete</th>
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</thead>
<tbody>
<tr>
<td>TWLTL Width</td>
<td>Des: 16 ft; Min: 14 ft</td>
<td>Des: 16 ft; Min: 12 ft</td>
<td>Des: 14 ft; Min: 11 ft</td>
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</tr>
<tr>
<td>Parking-Lane Width</td>
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</tr>
<tr>
<td>Median Width</td>
<td>Depressed</td>
<td>Existing</td>
<td>Des: 16 ft; Min: 2 ft</td>
<td>Des: 16 ft; Min: 2 ft</td>
<td>Des: 16 ft; Min: 2 ft</td>
<td></td>
</tr>
<tr>
<td>Raised Island</td>
<td>Des: 16 ft; Min: 2 ft</td>
<td>Des: 16 ft; Min: 2 ft</td>
<td>Des: 16 ft; Min: 2 ft</td>
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</tr>
<tr>
<td>Flush / Corrugated</td>
<td>55-4.05</td>
<td>55-4.05</td>
<td>55-4.05</td>
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<td></td>
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<tr>
<td>Sidewalk Width (11)</td>
<td>4 ft with 5 ft. Buffer (Des)</td>
<td>4 ft with 5 ft. Buffer (Des)</td>
<td>4 ft with 5 ft. Buffer (Des)</td>
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<td></td>
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</tr>
<tr>
<td>Bicycle-Lane Width (12)</td>
<td>Curbed: 5 ft</td>
<td>Curbed: 5 ft</td>
<td>Curbed: 5 ft</td>
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<tr>
<td>Obstruction-Free-Zone Width</td>
<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
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<tr>
<td>Typical Curbing Type, where used (13)</td>
<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
<td></td>
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</table>

**Note:** Des: Desirable; Min: Minimum

* Level One controlling criterion, see page 2 of 4.

GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, FOUR OR MORE LANES, 3R PROJECT

Figure 55-3E (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Built-Up</th>
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<tr>
<td><strong>Bridges</strong></td>
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<tr>
<td>New or Reconstructed Bridge</td>
<td>Ch. 403</td>
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<tr>
<td>*Structural Capacity (16)</td>
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<tr>
<td>Existing Bridge to Remain in Place</td>
<td>Ch. 72</td>
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<tr>
<td>*Structural Capacity</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>*Clear-Roadway Width</td>
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<td>Curbed: Full Approach Curb-to-Curb Width; Uncurbed: Travelway Plus 2 ft. on Each Side</td>
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<td></td>
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<tr>
<td>*Vertical Clearance, Arterial Under</td>
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<tr>
<td>New or Replaced Overpassing Bridge (18a &amp; 18c)</td>
<td>55-6.0</td>
<td>16.5 ft</td>
<td>16.5 ft (18b)</td>
<td>16.5 ft (18b)</td>
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<tr>
<td>Existing Overpassing Bridge (19)</td>
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<td>14.0 ft</td>
<td>14.0 ft</td>
<td>14.0 ft</td>
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<tr>
<td>Sign Truss / Pedestrian Bridge (18a &amp; 18c)</td>
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<td>Vertical Clearance, Arterial over Railroad (20)</td>
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<tr>
<td>Design Speed</td>
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<td>30 mph</td>
<td>35 mph</td>
<td>45 mph</td>
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<td>*Stopping Sight Distance, Desirable</td>
<td>55-4.02</td>
<td>200 ft</td>
<td>250 ft</td>
<td>360 ft</td>
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<td>Decision Sight Distance</td>
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<td>U: 620 ft</td>
<td>U: 720 ft</td>
<td>U: 930 ft</td>
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<tr>
<td>Speed / Path / Direction Change</td>
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<td>SU: 535 ft</td>
<td>SU: 625 ft</td>
<td>SU: 800 ft</td>
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<td>Stop Maneuver</td>
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<td>490 ft</td>
<td>590 ft</td>
<td>800 ft</td>
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<td>Intersection Sight Distance, -3% to +3% (21)</td>
<td>55-4.06</td>
<td>P: 355 ft</td>
<td>P: 415 ft</td>
<td>P: 530 ft</td>
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<td>SUT: 450 ft</td>
<td>SUT: 525 ft</td>
<td>SUT: 675 ft</td>
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<td>Alignment Elements</td>
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<td>*Minimum Radius</td>
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<td>See Section 55-4.03</td>
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<tr>
<td>*Superelavation Rate</td>
<td>55-4.03</td>
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<td>*Horizontal Sight Distance</td>
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<td>See Section 55-4.03</td>
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<tr>
<td>*Vertical Curvature, K-value</td>
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<tr>
<td>Crest</td>
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<tr>
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<tr>
<td>See Section 55-4.04</td>
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<tr>
<td>*Maximum Grade</td>
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<tr>
<td>Level</td>
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<td>10%</td>
<td>9%</td>
<td>8.5%</td>
</tr>
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<td>Rolling</td>
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<td>11%</td>
<td>10%</td>
<td>9.5%</td>
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<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
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<tr>
<td>Curbed Des: 0.5%; Curbed Min: 0.3%; Uncurbed: 0.0%</td>
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</tr>
</tbody>
</table>


* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets (the Green Book)* may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.
Design Forecast Period. For a partial 3R project, the pavement should be designed for at least a 10-year design life.

Design Speed. The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit on a non-posted highway. This is 30 mph, but with an engineering study it may be raised to a maximum of 55 mph.

On-Street Parking. In general, on-street parking is discouraged.

Travel Lane, Width. For an arterial on the National Truck Network, the right lane must be 12 ft in width. For a non-National-Truck-Network route, a minimum 11 ft travel lane should be used where truck volume exceeds 200 trucks per day. See Section 55-4.05.

Surface Type. The pavement type selection will be determined by the Office of Pavement Engineering or by the local jurisdiction.

Curb Offset. Vertical curbs which are either continuous or introduced intermittently may be offset 1 ft.

Shoulder Width. The value applies to paved-shoulder width. The following will also apply:
   a. For an uncurbed section, the shoulder is paved to the face of guardrail. The desirable guardrail offset is 2 ft from the usable-shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. If guardrail is present, the minimum offset from E.T.L. to face of guardrail should desirably be equal to the shy-line offset distance, but not less than 4 ft (see Section 49-4.0 for shy-line offsets). In a restrictive situation, the guardrail offset may be 0 ft from the usable-shoulder width.
   d. For a curbed section, the curb offset is included in the paved-shoulder width.

Cross Slope, Travel Lane. Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

Cross Slope, Shoulder. Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

Parking Lane Width. The following will apply:
   a. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be equal to the travel lane width plus the curb offset width (if present).
   b. A parking lane for residential usage may be 7 ft narrower.
   c. The cross slope for a parking lane is typically 1% steeper than that of the adjacent travel lane.

Sidewalk Width. Value is for the installation of a new sidewalk. An existing sidewalk width of 3 ft or greater (with or without a buffer) may be retained. A buffer strip of 4 ft or more is desirable.

Bicycle-Lane Width. The value is in addition to the width of a parking lane, if present. See Section 51-7.0 for additional details.
(13) **Curbing Type.** Vertical curbs may only be used with design speed lower than 50 mph.

(14) **Side Slopes.** Section 55-4.05 provides additional information for side slope criteria.

(15) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf desirably should be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(16) **Structural Capacity, New or Reconstructed Bridge.** The following will apply:
   a. Each State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. Each bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.

(17) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. The clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(18) **Vertical Clearance, Arterial Under Railroad.** The following will apply:
   a. Value includes an additional 6 in. allowance for a future pavement overlay.
   b. In a highly-urbanized area, a minimum clearance of 14.0 ft may be provided if there is at least one route with a 16.0-ft clearance.
   c. Vertical clearance applies from usable edge to usable edge of shoulder.

(19) **Vertical Clearance, Existing Bridge.** See Section 55-6.02 for additional information on minimum allowable vertical clearance.

(20) **Vertical Clearance, Arterial Over Railroad.** See Section 402-6.01(03) for additional information on railroad clearance under a highway.

(21) **Intersection Sight Distance.** For left turn onto a two-way, 4-lane undivided roadway. P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
# GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, TWO LINES, 3R PROJECT

**Figure 55-3F (Page 1 of 4)**

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Built-up</th>
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<tbody>
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<td><strong>Design Controls</strong></td>
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<tr>
<td>Design Forecast Period</td>
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<td>20 Years (1)</td>
<td>20 Years (1)</td>
</tr>
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<td>Posted Speed Limit</td>
<td>Posted Speed Limit</td>
<td>Posted Speed Limit</td>
</tr>
<tr>
<td>Access Control</td>
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<td>Partial Control / None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Des: B; Min: D</td>
<td>Des: C; Min: D</td>
<td>Des: C; Min: D</td>
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<td>On-Street Parking</td>
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<td>Optional (3)</td>
<td>Optional (3)</td>
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<tr>
<td><strong>Cross Section Elements</strong></td>
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</tr>
<tr>
<td>Travel Lane</td>
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<tr>
<td>*Width (4)</td>
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<td>Curbed: Des: 12 ft; Min: 11 ft</td>
<td>Curbed: Des: 12 ft</td>
</tr>
<tr>
<td>Uncurbed: Des: 12 ft; Min: 11 ft</td>
<td>Uncurbed: Des: 12 ft; Min: 11 ft</td>
<td>Curbed: Min: 10 ft</td>
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<td></td>
</tr>
<tr>
<td>Typical Surface Type (5)</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete</td>
<td>Asphalt / Concrete</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>*Curb Offset (6)</td>
<td>55-4.05</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
<td>Des: 2 ft; Min: 1 ft</td>
</tr>
<tr>
<td>Shoulder</td>
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</tr>
<tr>
<td>*Paved Width (7)</td>
<td>55-4.05</td>
<td>Curbed: Des: 10 ft; Min: 1 ft</td>
<td>Curbed: Des: 8 ft; Min: 1 ft</td>
<td>Des: 6 ft; Min: 2 ft</td>
</tr>
<tr>
<td>Uncurbed: Des: 10 ft; Min: 6 ft</td>
<td>Uncurbed: Des: 8 ft; Min: 4 ft</td>
<td>Des: 6 ft; Min: 2 ft</td>
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</tr>
<tr>
<td>Typical Surface Type (5)</td>
<td>Ch. 304</td>
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<td>Asphalt / Concrete</td>
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<tr>
<td>Cross Slope</td>
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</tr>
<tr>
<td>*Travel Lane (8)</td>
<td>55-4.05</td>
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<td>2%-3%</td>
<td>2%-3%</td>
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<tr>
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<td>Paved Width ≤ 4 ft: 2%-3%;</td>
<td>Paved Width ≤ 4 ft: 2%-3%;</td>
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<td>Paved Width &gt; 4 ft: 4%-6%</td>
<td>Paved Width &gt; 4 ft: 4%-6%</td>
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<td>Lane Width</td>
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<td>Des: 12 ft; Min: 10 ft</td>
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<td>Des: 1 ft; Min: 0 ft</td>
<td>Des: 1 ft; Min: 0 ft</td>
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<td>Des: 8 ft; Min: 2 ft</td>
<td>Des: 6 ft; Min: 2 ft</td>
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<td>Des: 16 ft; Min: 12 ft</td>
<td>Des: 16 ft; Min: 11 ft</td>
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<td>Des: 10 ft; Min: 8 ft (10)</td>
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<td>Curbed: 5 ft</td>
<td>Curbed: 5 ft</td>
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<td>See Section 55-5.02</td>
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<td>2:1 or Flatter (14)</td>
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<td>2:1 or Flatter (14)</td>
<td>2:1 or Flatter (14)</td>
<td>2:1 or Flatter (14)</td>
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Des: Desirable; Min: Minimum.
* Level One controlling criterion, see page 2 of 4
**GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, TWO LANES, 3R PROJECT**

*Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.*

<table>
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<th>Manual Section</th>
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<td>HL-93</td>
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<td>*Clear-Roadway Width (17)</td>
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<td>HS-20</td>
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<tr>
<td>*Structural Capacity</td>
<td>55-6.02</td>
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<td>*Clear-Roadway Width</td>
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<tr>
<td><strong>Vertical Clearance, Arterial Under</strong></td>
<td>44-4.0</td>
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<td>16.5 ft (18b)</td>
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<td>14 ft</td>
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<td>New: 17.5 ft; Existing: 17.0 ft</td>
<td>New: 17.5 ft; Existing: 17.0 ft</td>
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<td>Design Speed</td>
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<td>25 mph</td>
<td>30 mph</td>
<td>35 mph</td>
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<td>250 ft</td>
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<td>U: 620 ft</td>
<td>U: 720 ft</td>
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<td>500 ft</td>
<td>650 ft</td>
<td>825 ft</td>
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<td>See Section 55-4.03</td>
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<td>55-4.04</td>
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<tr>
<td>Crest</td>
<td>See Section 55-4.04</td>
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<td></td>
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<td>Sag</td>
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<td>55-4.04</td>
<td>11%</td>
<td>10%</td>
<td>9%</td>
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<td>11%</td>
<td>10%</td>
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</table>

(1) **Design Forecast Period.** For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(2) **Design Speed.** The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit on a non-posted highway. This is 30 mph, but with an engineering study it may be raised to a maximum of 55 mph.

(3) **On-Street Parking.** In general, on-street parking is discouraged.

(4) **Travel Lane, Width.** For an arterial on the National Truck Network, the right lane must be 12 ft in width. For a non-National-Truck-Network route, a minimum 11 ft travel lane should be used where truck volume exceeds 200 trucks per day. See Section 55-4.05.

(5) **Surface Type.** The pavement-type selection will be determined by the Office of Pavement Engineering or by the local jurisdiction.

(6) **Curb Offset.** The curb offset should be 2 ft. Vertical curbs which are either continuous or introduced intermittently may be offset 1 ft.

(7) **Shoulder Width.** The value applies to paved-shoulder width. The following will also apply:
   a. For an uncurbed section, the shoulder is paved to the face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. If guardrail is present, the minimum offset from E.T.L. to face of guardrail should desirably be equal to the shy-line offset distance, but not less than 4 ft (see Section 49-4.0 for shy-line offsets). In a restrictive situation, the guardrail offset may be 0 ft from the usable shoulder width.
   d. For a curbed section, the curb offset is included in the paved shoulder width.

(8) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(9) **Cross Slope, Shoulder.** Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

(10) **Parking-Lane Width.** The following will apply:
   a. Where the parking lane will be used as a travel lane during peak hours or may be converted to a travel lane in the future, the width should be equal to the travel lane width plus the curb offset width (if present).
   b. A parking lane for residential usage may be 7 ft narrower.
   c. The cross slope for a parking lane is typically 1% steeper than that for the adjacent travel lane. A buffer strip of 4 ft or wider is desirable.

(11) **Sidewalk Width.** Value is for the installation of a new sidewalk. An existing sidewalk width of 3 ft or greater (with or without a buffer) may be retained. A buffer strip of 4 ft or wider is desirable.

(12) **Bicycle-Lane Width.** The width is in addition to the width of parking lane, if present. See Section 51-7.0 for additional details.
(13) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.

(14) **Side Slopes, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf desirably should be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(16) **Structural Capacity, New or Reconstructed Bridge.** The following will apply:
   a. Each State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. Each bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.

(17) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. On a State highway, the minimum clear-roadway width should be 30 ft. Otherwise, the clear-roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(18) **Vertical Clearance, Arterial Under Railroad.** The following will apply:
   a. Value includes an additional 6 in. allowance for a future pavement overlay.
   b. In a highly-urbanized area, a minimum clearance of 14.0 ft may be provided if there is at least one route with a 16.0 ft clearance.
   c. Vertical clearance applies from usable edge to usable edge of shoulder.

(19) **Vertical Clearance, Existing Bridge.** See 55-6.02 for additional information on minimum allowable vertical clearance.

(20) **Vertical Clearance, Arterial Over Railroad.** See Section 402-6.01(03) for additional information on railroad clearance under a highway.

(21) **Intersection Sight Distance.** For left turn onto a 2-lane road, P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.

---

**Figure 55-3F GEOMETRIC DESIGN CRITERIA FOR URBAN ARTERIAL, TWO LANES, 3R PROJECT**

*Figure 55-3F (Page 4 of 4)*
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Built-Up</th>
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<td>Design Forecast Period</td>
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<td>20 Years (1)</td>
<td>20 Years (1)</td>
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<td>Posted Speed Limit</td>
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<td>Level of Service</td>
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<td>Desirable: C; Minimum: D</td>
<td>Desirable: C; Minimum: D</td>
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<td>Optional (3)</td>
<td>Optional (3)</td>
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<tr>
<td>Lane Width</td>
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<td>Des: 12 ft; Min: 9 ft</td>
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<td>Des: 16 ft; Min: 2 ft</td>
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<tr>
<td>Sidewalk Width (11)</td>
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<td>Des: 6 ft; Min: 4 ft</td>
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<td>See Section 55-5.02</td>
<td>See Section 55-5.02</td>
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<tr>
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Des: Desirable; Min: Minimum.

* Level One controlling criterion, see page 2 of 4

GEOMETRIC DESIGN CRITERIA FOR URBAN COLLECTOR, 3R PROJECT
Figure 55-3G (Page 1 of 4)
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<th>Manual Section</th>
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<td>HL-93</td>
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<td>Stop Maneuver</td>
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<td>430 ft</td>
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<td>See Section 55-4.05</td>
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<td></td>
<td></td>
<td>Sag</td>
<td>See Section 55-4.04</td>
<td></td>
</tr>
<tr>
<td></td>
<td>*Maximum Grade (21)</td>
<td>55-4.04</td>
<td>Level</td>
<td>11%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rolling</td>
<td>14%</td>
<td>13%</td>
</tr>
<tr>
<td></td>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>Curbed Des: 0.5%; Curbed Min: 0.3%</td>
<td>Uncurbed: 0.0%</td>
</tr>
</tbody>
</table>


* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s A Policy on Geometric Design of Highways and Streets (the Green Book) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.
Design Forecast Period. For a partial 3R project, the pavement should be designed for at least a 10 year design life.

Design Speed. The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit on a non-posted highway. This is 30 mph, but with an engineering study it may be raised to a maximum of 55 mph.

On-Street Parking. In general, on-street parking is discouraged.

Travel Lane, Width. A minimum 11 ft travel lane should be used where truck volume exceeds 200 trucks per day. See Section 55-4.05.

Surface Type. The pavement-type selection will be determined by the Office of Pavement Engineering or by the local jurisdiction.

Curb Offset. The curb offset should be 2 ft. Vertical curbs which are either continuous or introduced intermittently should be offset 1 ft.

Shoulder Width. The value applies to paved-shoulder width. The following will also apply:
   a. For an uncurbed section, the shoulder is paved to the face of guardrail. The desirable guardrail offset is 2 ft from the usable shoulder width. See Section 49-4.0 for more information.
   b. For an uncurbed section, a desirable additional 1 ft of compacted aggregate will be provided.
   c. If guardrail is present, the minimum offset from the E.T.L. to face of guardrail should desirably be equal to the shy-line offset distance, but not less than 4 ft (see Section 49-4.0 for shy-line offsets). In a restrictive situation, the guardrail offset may be 0 ft from the usable shoulder width.
   d. For a curbed section, the curb offset is included in the paved shoulder width.

Cross Slope, Travel Lane. Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

Cross Slope, Shoulder. Value is for a tangent section. See Figure 45-1A(1) or Figure 45-1A(2) for more-specific information. See Figure 43-3M or Figure 43-3N for shoulder cross slope on a horizontal curve.

Parking-Lane Width. A parking lane for residential usage may be 7 ft or less. The cross slope for a parking lane is typically 1% steeper than that for the adjacent travel lane. In a residential area, a parallel parking lane from 7 to 8 ft in width should be provided on one or both sides of the street. In a commercial or industrial area, the parking-lane width should range from 8 to 11 ft, and should usually be provided on both sides of the street. Where curb-and-gutter sections are used, the gutter-pan width may be considered as part of the parking-lane width. Where practical, the parking-lane width should be in addition to the gutter-pan width.

Sidewalk Width. Value is for the installation of a new sidewalk. An existing sidewalk width of 3 ft or greater (with or without a buffer) may be retained. A buffer strip of 4 ft or wider is more desirable.
(12) **Bicycle-Lane Width.** The width is in addition to the width of parking lane, if present. See Section 51-7.0 for additional details.

(13) **Curbing Type.** Vertical curbs may only be used with design speed lower than 50 mph.

(14) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.

(15) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf desirably should be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(16) **Structural Capacity, New or Reconstructed Bridge.** The following will apply:
   a. Each State-highway bridge within 15 mi of a Toll-Road gate must be designed for Toll-Road loading.
   b. Each bridge on an Extra-Heavy-Duty Highway must be designed for the Michigan Train truck loading configuration.

(17) **Width, New or Reconstructed Bridge.** See Section 402-6.02(01) for more information. On a State highway, the minimum clear-roadway width should be 30 ft. Otherwise, the clear roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(18) **Vertical Clearance, Collector Under Railroad.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulder.

(19) **Vertical Clearance, Existing Bridge.** See Section 55-6.02 for additional information on minimum allowable vertical clearance.

(20) **Vertical Clearance, Arterial Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(21) **Maximum Grades.** For a grade of less than 500 ft in length (PVT to PVC), a one-way downgrade, or a street with AADT < 400, the maximum grade may be 2% steeper than the value. Where adjacent sidewalks are present, the maximum desirable grade is 5%.

(22) **Intersection Sight Distance.** For left turn onto a 2-lane road, P = Passenger car; SUT = single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Design Values (By Type of Area)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Controls</td>
<td></td>
<td>Suburban</td>
</tr>
<tr>
<td>Design Forecast Period</td>
<td>55-4.01</td>
<td>20 Years (1)</td>
</tr>
<tr>
<td>*Design Speed, mph (2)</td>
<td>55-4.01</td>
<td>See Section 55-4.01</td>
</tr>
<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>None</td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Desirable: C; Minimum: D</td>
</tr>
<tr>
<td>On-Street Parking</td>
<td>45-1.0</td>
<td>Optional (3)</td>
</tr>
<tr>
<td>Travel Lane</td>
<td>55-4.05</td>
<td>Curbed: Des: 11 ft; Min: 10 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Uncurbed: Des: 11 ft; Min: 10 ft</td>
</tr>
<tr>
<td>Typical Surface Type</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete</td>
</tr>
<tr>
<td>*Curb Offset (5)</td>
<td>55-4.05</td>
<td>Des: 2 ft; Min: 1 ft</td>
</tr>
<tr>
<td>Shoulder</td>
<td>55-4.05</td>
<td>Curbed Des: 4 ft; Min: 1 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Uncurbed: Des: 4 ft; Min: 2 ft</td>
</tr>
<tr>
<td>Typical Surface Type</td>
<td>Ch. 304</td>
<td>Asphalt / Concrete / Aggregate / Earth</td>
</tr>
<tr>
<td>Cross Slope</td>
<td>55-4.05</td>
<td>2%-3%</td>
</tr>
<tr>
<td>Shoulder (7)</td>
<td>55-4.05</td>
<td>2%-3% Asphalt / Concrete; 6%-8% Aggregate; 8% Earth</td>
</tr>
<tr>
<td>Auxiliary Lane</td>
<td>55-4.05</td>
<td>Des: 11 ft; Min: 10 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Des: 1 ft; Min: 0 ft</td>
</tr>
<tr>
<td>Shoulder Width</td>
<td>55-4.05</td>
<td>Des: 4 ft; Min: 1 ft</td>
</tr>
<tr>
<td>Typical Surface Type</td>
<td>Ch. 52</td>
<td>Asphalt / Concrete / Aggregate / Earth</td>
</tr>
<tr>
<td>Parking-Lane Width (3)</td>
<td>45-1.04</td>
<td>Des: 9 ft; Min: 7 ft</td>
</tr>
<tr>
<td>Sidewalk Width (8)</td>
<td>55-4.05</td>
<td>4 ft with 5 ft Buffer (Des)</td>
</tr>
<tr>
<td>Bicycle-Lane Width (9)</td>
<td>51-7.0</td>
<td>Curbed: 5 ft</td>
</tr>
<tr>
<td>Obstruction-Free-Zone Width</td>
<td>55-5.02</td>
<td>See Section 55-5.02</td>
</tr>
<tr>
<td>Typical Curbing Type, where used (5)</td>
<td>55-4.05</td>
<td>Vertical / Sloping</td>
</tr>
<tr>
<td>Side Slopes, Uncurbed</td>
<td>Cut</td>
<td>2:1 or Flatter (10)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(10)</td>
</tr>
<tr>
<td></td>
<td>Ditch Width</td>
<td>2:1 or Flatter (10)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(10)</td>
</tr>
<tr>
<td></td>
<td>Backslope</td>
<td>2:1 or Flatter (10)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(11)</td>
</tr>
<tr>
<td>Side Slopes, Curbed</td>
<td>Cut, Backslope</td>
<td>2:1 or Flatter (10)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(11)</td>
</tr>
</tbody>
</table>

Des: Desirable; Min: Minimum.

* Level One controlling criterion, see page 2 of 4

GEOMETRIC DESIGN CRITERIA FOR URBAN LOCAL STREET, 3R PROJECT

Figure 55-3H (Page 1 of 4)
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Manual Section</th>
<th>Suburban</th>
<th>Intermediate</th>
<th>Built-Up</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>New or Reconstructed Bridge</strong></td>
<td><em>Structural Capacity</em></td>
<td>Ch. 403</td>
<td>HL-93</td>
<td>HL-93</td>
</tr>
<tr>
<td></td>
<td><em>Clear-Roadway Width</em></td>
<td>55-6.03</td>
<td>Curbed: Full Approach Curb-to-Curb Width Uncurbed: (12)</td>
<td></td>
</tr>
<tr>
<td><strong>Existing Bridge to Remain in Place</strong></td>
<td><em>Structural Capacity (13)</em></td>
<td>Ch. 72</td>
<td>HS-15</td>
<td>HS-15</td>
</tr>
<tr>
<td></td>
<td><em>Clear-Roadway Width</em></td>
<td>55-6.02</td>
<td>Existing Width (14)</td>
<td></td>
</tr>
<tr>
<td><strong>New or Replaced Overpassing Bridge (15)</strong></td>
<td>44-4.0</td>
<td>14.5 ft</td>
<td>14.5 ft</td>
<td>14.5 ft</td>
</tr>
<tr>
<td><strong>Existing Overpassing Bridge (16)</strong></td>
<td>14.0 ft</td>
<td>14.0 ft</td>
<td>14.0 ft</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Clearance, Local over Railroad (17)</strong></td>
<td>23.0 ft</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Speed</th>
<th>25 mph</th>
<th>30 mph</th>
<th>35 mph</th>
<th>45 mph</th>
<th>50 mph</th>
<th>55 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Stopping Sight Distance, Desirable</em></td>
<td>155 ft</td>
<td>200 ft</td>
<td>250 ft</td>
<td>360 ft</td>
<td>425 ft</td>
<td>495 ft</td>
</tr>
<tr>
<td><strong>Decision Sight Distance</strong></td>
<td>Speed / Path / Direction Change</td>
<td>42-2.0</td>
<td>U: 515 ft</td>
<td>U: 620 ft</td>
<td>U: 720 ft</td>
<td>U: 930 ft</td>
</tr>
<tr>
<td><strong>Stop Maneuver</strong></td>
<td>430 ft</td>
<td>490 ft</td>
<td>590 ft</td>
<td>800 ft</td>
<td>910 ft</td>
<td>1030 ft</td>
</tr>
<tr>
<td><strong>Intersection Sight Distance, -3% to +3% (18)</strong></td>
<td>P: 280 ft</td>
<td>P: 330 ft</td>
<td>P: 390 ft</td>
<td>P: 500 ft</td>
<td>P: 550 ft</td>
<td>P: 610 ft</td>
</tr>
<tr>
<td><strong>Minimum Radius</strong></td>
<td>55-4.03</td>
<td>See Section 55-4.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Superelevation Rate</strong></td>
<td>55-4.03</td>
<td>See Section 55-4.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Horizontal Sight Distance</strong></td>
<td>55-4.03</td>
<td>See Section 55-4.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Curvature, K-value</strong></td>
<td>Crest</td>
<td>55-4.04</td>
<td>See Section 55-4.04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sag</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Maximum Level</strong></td>
<td>55-4.04</td>
<td>In a residential area, the maximum grade should not exceed 15%. In an industrial or a commercial area, the maximum grade should not exceed 8%.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Minimum Grade</strong></td>
<td>55-4.04</td>
<td>Curbed Des: 0.5%; Curbed Min: 0.3% Uncurbed: 0.0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


* Level One controlling criterion. Except as noted in this chapter, the values shown in AASHTO’s *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) may be used as minimum values if they are lower than similar values shown herein. A controlling criterion that does not meet the minimum value is a design exception and is subject to approval. A streamlined design exception may be used for 3R projects. See Section 40-8.0.

This figure applies only to a federal-aid funded project.
(1) **Design Forecast Period.** For a partial 3R project, the pavement should be designed for at least a 10 year design life.

(2) **Design Speed.** The minimum design speed should equal the anticipated posted speed limit after construction or the legal speed limit on a non-posted highway. This is 30 mph, but with an engineering study it may be raised to a maximum of 55 mph.

(3) **On-Street Parking.** In general, on-street parking is discouraged. However, if parking lanes are used, cross slopes are typically 1% steeper than that of the adjacent travel lane. In a residential area, a parallel parking lane from 7 to 8 ft in width should be provided on one or both sides of the street. In a commercial or industrial area, parking-lane width should range from 8 to 11 ft, and should usually be provided on both sides of the street. Where curb-and-gutter sections are used, the gutter-pan width may be considered as part of the parking-lane width. Where practical, the parking-lane width should be in addition to the gutter-pan width.

(4) **Travel Lane, Width.** A minimum 11 ft travel lane should be used where truck volume exceeds 200 trucks per day. See Section 55-4.05.

(5) **Curb Offset.** A vertical-curb offset should be 2 ft. Vertical curbs which are either continuous or introduced intermittently may be offset 1 ft. A sloping-curb offset may be zero. For a curbed section, the curb offset is included in the paved shoulder width. Vertical curbs may only be used with design speed lower than 50 mph.

(6) **Cross Slope, Travel Lane.** Cross slopes of 1.5% are acceptable on an existing bridge to remain in place.

(7) **Cross Slope, Shoulder.** Value is for a tangent section. See Section 43-3.06 for shoulder cross slopes on a horizontal curve.

(8) **Sidewalk Width.** Value is for the installation of a new sidewalk. An existing sidewalk width of 3 ft or greater (with or without a buffer) may be retained. A buffer strip of 4 ft or wider is desirable.

(9) **Bicycle-Lane Width.** The width is in addition to the width of parking lane, if present. See Section 51-7.0 for additional details.

(10) **Side Slopes.** Section 55-4.05 provides additional information for side-slope criteria.
(11) **Side Slope, Curbed, Cut.** A shelf or sidewalk will be present immediately behind the curb before the toe of the backslope. The minimum width of a shelf desirably should be 6 ft. Where a sidewalk is present, the toe of the backslope will be 1 ft beyond the edge of sidewalk. See Section 45-3.0 for more information.

(12) **Width, New or Reconstructed Bridge.** See Section 402-6.2(01) for more information. The clear roadway width is the algebraic sum of the following:
   a. the approach traveled way width;
   b. the approach usable shoulder width without guardrail; and
   c. a bridge-railing offset (see Figure 402-6H).

(13) **Structural Capacity, Existing Bridge to Remain in Place.** For a street with AADT \( \leq 50 \), an HS-10 loading is acceptable.

(14) **Width, Existing Bridge to Remain in Place.** If the width of the existing bridge is less than the approach travelway width, the bridge should be widened to at least the travelway width.

(15) **Vertical Clearance, Local Under.** Value includes an additional 6 in. allowance for a future pavement overlay. Vertical clearance applies from usable edge to usable edge of shoulder.

(16) **Vertical Clearance, Existing Bridge.** See Section 55-6.02 for additional information on minimum allowable vertical clearance.

(17) **Vertical Clearance, Local Over Railroad.** See Chapter 402-6.01(03) for additional information on railroad clearance under a highway.

(18) **Intersection Sight Distance.** For left turn onto a 2 lane road, \( P = \) Passenger car; \( SUT = \) single unit truck. See Figure 46-10G for value for a combination truck.
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Calculated $K$ Value ((K = \frac{V^2}{46.5}))</th>
<th>$K$ Value Rounded For Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>8.6</td>
<td>9</td>
</tr>
<tr>
<td>25</td>
<td>13.4</td>
<td>14</td>
</tr>
<tr>
<td>30</td>
<td>19.4</td>
<td>20</td>
</tr>
<tr>
<td>35</td>
<td>26.3</td>
<td>27</td>
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<td>45</td>
<td>43.5</td>
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<td>50</td>
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<td>60</td>
<td>77.4</td>
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<tr>
<td>70</td>
<td>105.4</td>
<td>106</td>
</tr>
<tr>
<td>75</td>
<td>121.0</td>
<td>122</td>
</tr>
</tbody>
</table>

$$L = \frac{AV^2}{46.5} = KA$$

Where:
- $L$ = Length of vertical curve, ft.
- $A$ = Algebraic difference between grades, %
- $K$ = Horizontal distance required to effect a 1% change in gradient
- $V$ = Design speed, mph

**$K$ Value for Sag Vertical Curve**
(Comfort Criteria — 3R Project)

Figure 55-4A
<table>
<thead>
<tr>
<th>Span</th>
<th>Rise</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 10 ft</td>
<td>All</td>
<td>B preferred; A acceptable</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>&lt; 5.5 ft</td>
<td>B preferred; A acceptable</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>≥ 5.5 ft</td>
<td>B</td>
</tr>
</tbody>
</table>

A Provide a clear zone with 6:1 slopes or flatter at least a distance $L_r$ in advance of, and 100 ft beyond, the structure. Taper 10:1 on both sides of the structure to tie back in.

B Guardrail should be placed. Use treatment A if guardrail is impractical due to the close proximity of a public road approach or drive. The drive grade should be designed to be compatible with the clear-zone slope. The drive sideslope should be 10:1.

CLEAR ZONE / GUARDRAIL AT CULVERT

Figure 55-5A(1)
<table>
<thead>
<tr>
<th>Design Speed, mph</th>
<th>AADT &lt; 1,000</th>
<th>1,000 ≤ AADT &lt; 5,000</th>
<th>5,000 ≤ AADT &lt; 10,000</th>
<th>AADT ≥ 10,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>220</td>
<td>260</td>
<td>299</td>
<td>361</td>
</tr>
<tr>
<td>60</td>
<td>171</td>
<td>181</td>
<td>210</td>
<td>260</td>
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<td>55</td>
<td>151</td>
<td>164</td>
<td>190</td>
<td>237</td>
</tr>
<tr>
<td>50</td>
<td>131</td>
<td>151</td>
<td>171</td>
<td>210</td>
</tr>
<tr>
<td>45</td>
<td>115</td>
<td>131</td>
<td>151</td>
<td>184</td>
</tr>
<tr>
<td>40</td>
<td>99</td>
<td>112</td>
<td>131</td>
<td>161</td>
</tr>
<tr>
<td>30</td>
<td>69</td>
<td>79</td>
<td>89</td>
<td>112</td>
</tr>
</tbody>
</table>

**RUNOUT LENGTH, \( L_R \) (ft) FOR RESTRICTIVE CONDITION**

*Figure 55-5B*
<table>
<thead>
<tr>
<th>Spot, Section, or Intersection</th>
<th>No. Accidents in Year No.</th>
<th>No. V</th>
<th>No. I</th>
<th>No. F</th>
<th>Accident Codes from IDM Figure 55-8B</th>
<th>Reference Route No.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
</tbody>
</table>

V = Vehicles; I = Injured; F = Fatalities

ACCIDENT ANALYSIS FORM

Figure 55-8A
### ACCIDENT-ANALYSIS FORM CODES

**Figure 55-8B**
Note: Number Codes apply to Figure 55-5B.

COLLISION DIAGRAM CODES

Figure 55-8C
<table>
<thead>
<tr>
<th>DRIVER-RELATED</th>
<th>Correctable By Safety Enhancement</th>
<th>Not Correctable By Safety Enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsafe Speed</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Sick</td>
<td>--</td>
<td>X</td>
</tr>
<tr>
<td>Failed to Yield R/W</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Fell Asleep</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Following Too Close</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Lost Consciousness</td>
<td>--</td>
<td>X</td>
</tr>
<tr>
<td>Improper Passing</td>
<td>X</td>
<td>--</td>
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<tr>
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<td>Physical Disability</td>
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<tr>
<td>Drug Involvement</td>
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| VEHICLE-RELATED                     |                                   |                                       |
| Brakes Defective                    | X                                 | X                                     |
| Tow Hitch Defective                 | --                                | X                                     |
| Headlights Defective                | X                                 | X                                     |
| Over or Improper Load               | --                                | X                                     |
| Other Lighting Defects              | X                                 | X                                     |
| Oversized Load on Vehicle           | --                                | X                                     |
| Steering Failure                    | --                                | X                                     |
| Tire Failure/Inadequate             | X                                 | X                                     |

| ENVIRONMENT-RELATED                 |                                   |                                       |
| Animal on Roadway                   | X                                 | --                                    |
| Holes/Deep Ruts/Bumps               | X                                 | --                                    |
| Glare                               | X                                 | X                                     |
| Road Under Const. / Maint.          | --                                | --                                    |
| View Obstructed/Limited             | X                                 | X                                     |
| Debris in Roadway                   | --                                | X                                     |
| Improperly Parked Vehicle(s)        | X                                 | X                                     |
| Improper/Non-Working Traffic Ctrl(s)| X                                 | X                                     |
| Fixed Object(s)                     | X                                 | X                                     |
| Slippery Surface                    | X                                 | --                                    |
| Shoulders Defective                 | X                                 | --                                    |
| Water Ponding                       | X                                 | --                                    |
| Roadside Hazards                    | X                                 | --                                    |

**CONTRIBUTING CIRCUMSTANCES**

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<td>Run-off-roadway</td>
<td>Slippery Pavement</td>
<td>Improve skid resistance; provide adequate drainage; groove existing pavement</td>
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<td>Roadway Design</td>
<td>Widen lanes/shoulders; relocate islands; provide proper super-elevation; install/improve traffic barriers; improve alignment/grade; flatten slopes/ditches; provide escape ramp</td>
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<td>Inadequate for Traffic Conditions</td>
<td>Improve/install pavement markings; install roadside delineators; install advance warning lights</td>
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<td>Poor Delineation</td>
<td>Improve/install pavement markings; install roadside delineators; install advance warning lights</td>
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<td></td>
<td>Poor Visibility</td>
<td>Improve roadway lighting; increase sign size</td>
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<td></td>
<td>Inadequate Shoulder</td>
<td>Upgrade roadway shoulders</td>
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<td>Improper Channelization</td>
<td>Improve channelization</td>
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<tr>
<td>Overturn</td>
<td>Roadside Features</td>
<td>Flatten slopes and ditches; relocate drainage facilities; extend culverts; provide traversable culvert end treatments; install/improve traffic barriers</td>
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<td>Widen lane/shoulder; upgrade shoulder surface; remove curbing obstructions; revise cross slope</td>
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<td>Pavement Feature</td>
<td>Eliminate edge dropoff; improve superelevation/crown</td>
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<td>Accident Pattern</td>
<td>Probable Cause</td>
<td>Safety Enhancement</td>
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<tr>
<td>------------------</td>
<td>----------------</td>
<td>--------------------</td>
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<tr>
<td>Bridges</td>
<td>Alignment</td>
<td>Realign bridge/roadway; install advance warning signs; improve delineation/markings</td>
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<td>Narrow Roadway</td>
<td>Widen structure; improve delineation/markings; install signing/Signals</td>
</tr>
<tr>
<td></td>
<td>Visibility</td>
<td>Remove obstruction; install advance warning signs; improve delineation and markings</td>
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<tr>
<td></td>
<td>Vertical Clearance</td>
<td>Rebuild structure/adjust roadway grade; install advance warning signs; improve delineation and markings; provide height restrictor/warning device</td>
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<tr>
<td></td>
<td>Slippery Surface (Wet/Icy)</td>
<td>Resurface deck; improve skid resistance; provide adequate drainage; provide special signing</td>
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<tr>
<td></td>
<td>Rough Surface</td>
<td>Resurface deck; rehabilitate joints; regrade approaches</td>
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<tr>
<td></td>
<td>Inadequate Barrier</td>
<td>Upgrade bridge rail system; upgrade approach rail/terminals; upgrade bridge - approach rail connections; remove hazardous curb; improve delineation and marking</td>
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<td>Parked Vehicles</td>
<td>Widen lanes/shoulders</td>
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**ACCIDENT ANALYSIS**

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<th>Probable Cause</th>
<th>Safety Enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fixed Object</strong></td>
<td>Obstructions In or Too Close to Roadway</td>
<td>Remove/relocate obstacles; install breakaway features to light poles, signposts, etc.; protect objects with guardrail; install crash cushions; delineation/reflectorize safety hardware</td>
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<td></td>
<td>Inadequate Lighting</td>
<td>Improve roadway lighting</td>
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<td></td>
<td>Inadequate Pavement</td>
<td>Install reflectorized pavement marking lines/raised markers</td>
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<td></td>
<td>Inadequate Signs, Delineators and Guardrail</td>
<td>Install reflectorized paint and/or reflectors on the obstruction; add special signing; upgrade barrier system</td>
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<tr>
<td></td>
<td>Inadequate Road Design</td>
<td>Improve alignment/grade; provide proper superelevation; install warning signs/delineators; provide wider lanes</td>
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<tr>
<td></td>
<td>Slippery Surface</td>
<td>Improve skid resistance; provide adequate drainage; groove existing pavement</td>
</tr>
<tr>
<td><strong>Sideswipe or Head-On</strong></td>
<td>Inadequate Road Design</td>
<td>Provide wider lanes; improve alignment/grade; provide passing lanes; provide roadside delineators; sign and mark unsafe passing areas</td>
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<tr>
<td></td>
<td>Inadequate Shoulder</td>
<td>Improve shoulders</td>
</tr>
<tr>
<td></td>
<td>Excessive Vehicle Speed</td>
<td>Install median devices</td>
</tr>
<tr>
<td></td>
<td>Inadequate Pavement Markings</td>
<td>Install/improve centerlines, lane lines and edgelines; install reflectorized markers</td>
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<tr>
<td></td>
<td>Inadequate Channelization</td>
<td>Install acceleration and deceleration lanes; improve/install channelization; provide turning bays</td>
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<tr>
<td></td>
<td>Inadequate signing</td>
<td>Provide advance direction and warning signs; add illuminated name signs</td>
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</table>

ACCIDENT ANALYSIS

Figure 55-8E (Continued)
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<thead>
<tr>
<th>Accident Pattern</th>
<th>Probable Cause</th>
<th>Safety Enhancement</th>
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<tbody>
<tr>
<td>Intersection (Signalized/ Unsignalized) Left-Turn, Head-On, Right Angle, Rear End</td>
<td>Large Volume of Left/ Right Turns</td>
<td>Widen road; channelize intersection; install stop signs; provide signal; increase curb radii</td>
</tr>
<tr>
<td>Restricted Sight Distance</td>
<td>Remove sight obstructions; provide adequate channelization; provide left/ right turn lanes; install warning signs; install stop signs; install signal; install advance markings to supplement signs; install stop bars</td>
<td></td>
</tr>
<tr>
<td>Slippery Surface</td>
<td>Improve skid resistance; provide adequate drainage; groove pavement</td>
<td></td>
</tr>
<tr>
<td>Large Numbers of Turning Vehicles</td>
<td>Create left- or right-turn lanes; curb radii; install signal</td>
<td></td>
</tr>
<tr>
<td>Inadequate Lighting</td>
<td>Improve roadway lighting</td>
<td></td>
</tr>
<tr>
<td>Lack of Adequate Gaps</td>
<td>Provide signal; provide stop signs</td>
<td></td>
</tr>
<tr>
<td>Crossing Pedestrians</td>
<td>Install/improve signing or marking of pedestrian crosswalks; install signal</td>
<td></td>
</tr>
<tr>
<td>Large Total Intersection Volume</td>
<td>Install signal; add traffic lane</td>
<td></td>
</tr>
<tr>
<td>Excessive Speed on Approaches</td>
<td>Install rumble strips; improve warning devices</td>
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</tr>
<tr>
<td>Inadequate Advance Warning Signs</td>
<td>Install advance warning signs</td>
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<tr>
<td>Inadequate Traffic Control Devices</td>
<td>Upgrade traffic control devices</td>
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</tr>
<tr>
<td>Poor Visibility of Signals</td>
<td>Install/improve advance warning devices; install overhead signals; install 305-mm signal lenses; install visors/back plates; relocate signals; remove sight obstructions; add illuminated/reflectorized name signs</td>
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<tr>
<td>Un warranted Signals</td>
<td>Remove signals</td>
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<tr>
<td>Inadequate Signal Timing</td>
<td>Upgrade signal system timing/phasing</td>
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**ACCIDENT ANALYSIS**

Figure 55-8E (Continued)
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<thead>
<tr>
<th>Accident Pattern</th>
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<tbody>
<tr>
<td>Access Related</td>
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<tr>
<td>Left-Turning Vehicles</td>
<td>Install median devices; install two-way left-turn lanes</td>
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</tr>
<tr>
<td>Improperly Located Driveway</td>
<td>Move driveway to side street; install curbing to define driveway location; consolidate adjacent driveways</td>
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</tr>
<tr>
<td>Right-Turning Vehicles</td>
<td>Provide right-turn lanes; increase the width of driveways; widen through lanes; increase curb radii</td>
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</tr>
<tr>
<td>Large Volume of Through Traffic</td>
<td>Move driveway to side street; construct a local service road</td>
<td></td>
</tr>
<tr>
<td>Large Volume of Driveway Traffic</td>
<td>Signalize driveway; provide acceleration and deceleration lanes; channelize driveway</td>
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</tr>
<tr>
<td>Restricted Sight</td>
<td>Remove obstructions</td>
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<tr>
<td>Inadequate Lighting</td>
<td>Improve street lighting</td>
<td></td>
</tr>
<tr>
<td>Nighttime</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor Visibility or Lighting</td>
<td>Install/improve streetlighting; install/improve delineation/markings; install/improve warning signs</td>
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</tr>
<tr>
<td>Poor Sign Quality</td>
<td>Upgrade signing; provide illuminated reflectorized signs</td>
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</tr>
<tr>
<td>Inadequate Channelization or Delineation</td>
<td>Install pavement markings; improve channelization/delineation</td>
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**ACCIDENT ANALYSIS**

**Figure 55-8E (Continued)**
<table>
<thead>
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<th>Accident Pattern</th>
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<td>Pedestrian/Bicycle</td>
<td>Limited Sight Distance</td>
<td>Remove sight obstructions; install/improve pedestrian crossing signs and markings</td>
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<td>Inadequate Protection</td>
<td>Add pedestrian refuge islands</td>
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<td>Inadequate Signals/Signs</td>
<td>Install/upgrade signals/signs</td>
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<td>Mid-block Crossings</td>
<td>Install warning signs/markings</td>
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<td>Inadequate Pavement Markings</td>
<td>Supplement markings with signing; upgrade pavement markings</td>
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<td>Lack of Crossing Opportunity</td>
<td>Install traffic/pedestrian signals; install pedestrian crosswalk and signs</td>
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<td>Inadequate Lighting</td>
<td>Improve lighting</td>
</tr>
<tr>
<td></td>
<td>Excessive Vehicle Speed</td>
<td>Install proper warning signs</td>
</tr>
<tr>
<td></td>
<td>Pedestrian/Bicycles on Roadway</td>
<td>Install sidewalks; install bike lanes/path; eliminate roadside obstructions; install curb ramps</td>
</tr>
<tr>
<td></td>
<td>Long Distance to Nearest Crosswalk</td>
<td>Install pedestrian crosswalk; install pedestrian actuated signals</td>
</tr>
<tr>
<td>Wet Pavement</td>
<td>Slippery Pavement</td>
<td>Improve skid resistance; groove existing pavement</td>
</tr>
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<td>Inadequate Drainage</td>
<td>Provide adequate drainage</td>
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<td></td>
<td>Inadequate Pavement Markings</td>
<td>Install raised/reflectorized pavement markings</td>
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**ACCIDENT ANALYSIS**

Figure 55-8E (Continued)
<table>
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<td>Railroad Crossings</td>
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<td>Remove sight obstructions; reduce grade; install active warning devices; install advance warning signs</td>
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<td>Poor Visibility</td>
<td>Improve roadway lighting; increase size of signs</td>
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<td>Inadequate Pavement Markings</td>
<td>Install advance markings to supplement signs; install stop bars; install/improve pavement markings</td>
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<td>Rough Crossing Surface</td>
<td>Improve crossing surface</td>
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<td>Sharp Crossing Angle</td>
<td>Rebuild crossing with proper angle</td>
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**ACCIDENT ANALYSIS**

Figure 55-8E (Continued)
CHAPTER 56

Partial 3R Projects

NOTE: This chapter is currently being re-written and its content will be included in Chapter 302 in the future.

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<td>56-2.0, 56-4.06(06), 56-4.09(01), 56-4.09(02), Figure 56-4F</td>
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<td>Jul. 2016</td>
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CHAPTER 56

PARTIAL 3R PROJECTS

56-1.0 INTRODUCTION

Many highways cannot satisfy the values for full Resurfacing, Restoration, and Rehabilitation (3R) criteria. For these reasons, the Department has adopted a partial 3R concept with different limits for geometric design criteria for pavement rehabilitation on an existing highway. This is due to constraints in right-of-way, construction time, pavement conditions, or cost. The criteria for rehabilitation of an existing highway is based on sound engineering practices, experience, and assessment of the basic principles of geometric design and how the criteria for other types of construction can be adapted to an existing highway within practical limits. The goal of partial 3R is to preserve and maintain the existing highway system.

This chapter provides the Department’s criteria and guide to the development of a partial 3R project. The objective of this chapter is to unify and standardize the project development, design criteria, field data collection, and final presentation of plans and related documents used in the development of a partial 3R project. This chapter should not be interpreted as specifications and should not replace sound judgment. The designer should refer to the INDOT Standard Drawings or the Standard Specifications to resolve questions concerning materials, standard details, workmanship, pay items, pay units, etc.

See Chapter 52, or the AASHTO Policy for Geometric Design of Highways and Streets for additional information.

Where a partial 3R project scope of work includes costly items such as bridge reconstruction or replacement or major alignment corrections which have a long service life, the project should be returned to the Engineering Assessment stage, for consideration as 3R, 4R, or possibly new construction.

56-2.0 ENGINEERING ASSESSMENT [Rev. Feb. 2014]

The district Office of Design, in cooperation with the Planning Division and its Office of Pavement Engineering, will determine the need for and propose partial 3R work on a given route. The district will make a recommendation and justification regarding the type of partial 3R project. The recommendation will be reviewed by the Planning Division’s Safety Team and...
Mobility Team. The Teams will use their Pavement Management System to determine the needs of the pavement and to document the condition of the pavement prior to approving the appropriate type of treatment. The Teams will then discuss their findings with the district. The Planning Division and the district are to ultimately concur in whether the partial 3R project should be designed as a preventative maintenance, functional, or structural treatment, or instead, a full 3R project. The Safety Team leader may make recommendations relative to highway safety needs. The Planning Division then authorizes the project. This information is provided to the district’s Office of Design, which then begins the design process.

Right-of-way acquisition should not be required for partial 3R work.

Partial 3R work deemed an alteration must include curb ramps in the scope of work. See Section 51-1.04.

The following defines the scope of work to be performed for each type of partial 3R project.

56-2.01 Types of Partial 3R Projects

56-2.01(01) Preventative Maintenance Treatment

Preventative maintenance consists of a pavement-surface treatment used to preserve and extend the service life of the pavement. It should be designed in accordance with Section 52-7.04(01).

56-2.01(02) Functional Treatment

A functional treatment should be used to correct pavement deficiencies such as roughness or poor frictional properties. The intent is to improve the roadway serviceability by correcting distresses caused by traffic or environmental conditions. It should be designed in accordance with Section 52-7.04(02).

56-2.01(03) Structural Treatment

A structural treatment should be used where the existing pavement structure has failed due to a load-related distress. It should be designed in accordance with Section 52-7.04(03).
56-2.02 Analysis of Accident Data

Accident data should be analyzed in accordance with Section 55-5.01, except that a formal report is not necessary. Each location with a definite accident pattern should be indicated on the accident-data computer printout. Spot-improvements work at such a location should either be incorporated into the project or programmed separately.

56-2.03 Project Classification

Classification of work as partial 3R, full 3R, partial 4R, or full 4R must be determined in accordance with Section 40-6.0.

56-3.0 PRELIMINARY DESIGN PROCESS

Preliminary project parameters and criteria are discussed and outlined below.


The designer should review the existing project files, plans, and resurface-contract documents, if applicable, for additional information. Such plans include original stationing, roadbed characteristics, structure information, and original drainage patterns. Previous resurface-contract documents can include valuable supplemental information. See Section 5-2.02 for information regarding traffic data, traffic forecasting, and requesting crash data.

56-3.02 Preliminary Project Schedule

The designer should prepare a preliminary schedule with estimated completion dates for the following key activities or mileposts.

005  Project Started
040  Project Scope Complete
060  Start Plan Development
075  Survey Complete
085  Preliminary Field Check
110  Geotechnical Investigation (if required)
115  Final Pavement Design Approval
56-3.03 Environmental Document

The designer should verify the need for an environmental document and identify the required environmental permits. Each project requires a level of environmental documentation. For a partial 3R project, such documentation consists of a Categorical Exclusion. A Categorical Exclusion is described in Section 7-1.01(01).

56-3.04 Bridge Structure Considerations

Where a bridge structure is encountered within the project limits, the designer should consult with the district bridge engineer concerning needed improvements or needed repairs. A memorandum should be written to the Planning Division’s Bridge Management Team with a copy to the Production Management Division’s Bridge Rehabilitation and Ratings Team. The memorandum should provide details about the proposed project, and should request their comments and recommendations. The memorandum should address proposed milling, spot pavement replacement, horizontal and vertical clearances if they are factors, weight restrictions, and all other factors which can affect the structures.

56-3.05 Unusual Soil Conditions

If there are indications of peat deposits, rock outcroppings, or other unusual soil conditions, long-term repair of such items should be programmed separately through INSTIP.

56-3.06 Stationing

Stationing should match the existing plans where possible. If the project limits extend beyond the stationing limits of the existing plans, the stationing should be extended to include the project limits. Stationing should refer to the Reference Post System (RPS). For known features, see the Physical Features Inventory. Stations should be marked onto the pavement with traffic paint rather than spray paint. Stationing marking options are shown in Figure 56-3A.
If new English-units stationing must be used, it should have an assumed starting station of 100+00.

56-3.07 Field Notes

The designer should ascertain that all field data is adequate to design the project. See Figure 56-3B, Collection of Field Data, for forms and format. Field notes should be collected in the form of a strip map showing all existing details including, but not limited to, intersecting roads, drives, and railroads, pipe structures, headwalls, curbs, manholes, survey monuments, guardrail, traffic detector loops, stop lines, crosswalk lines, raised pavement markers, areas of grading, patching, milling, utilities in the area, or other specialty items. All items shown in the field notes should have a station and offset reference. Field notes should begin at the bottom of the page and proceed upstation. It is not necessary to strip-map sections of the project which are consistent and for which no work, other than paving, is specified. For this situation, a note reading, Consistent section from Station ____ to Station ____. should be shown, thence continuing with the stationing. Cross sections should be collected in a survey level field book, or in an approved electronic format. Cross sections may include sections for pipe replacement, linear grading, drainage, profiles, or other miscellaneous information.

If a field book is used, the front cover of the field book should be labeled as shown in the example as follows:

FIELD NOTES
DES NO. ________
(route)
FROM ___ mi (dir.) of (route) TO ___ mi (dir.) of (route)
RP ___+__ TO RP ___+
____________ DISTRICT

The data collector should collect the field data in a logical manner. Information regarding the content of the field book is shown in Chapters 22 and 23. The procedure for collecting field notes is as follows.

1. Walk the roadway to ascertain that everything is logged.
2. By category, count all mailbox approaches, field entrances, commercial drives, etc. Those with unusual sizes or shapes should be located by station and offset, and the dimensions noted in detail.

3. Indicate locations of underdrains and outlets.

4. Indicate pavement and shoulder types and width changes by station or dimensions.

5. Indicate all of the information required relative to the items listed on the Review of Traffic Items.

6. The district Highway Management Division, prior to the field survey, should have furnished a list of structures to be replaced, side ditches to be cleaned, etc. All dimensions, elevations, or other information needed to design the changes in these items should be gathered at this time.

7. If gabions or riprap are required, obtain all of the data necessary to incorporate them into the design.

8. If new or additional guardrail is required, information should be collected for design.

9. If an outcropping is to be removed, gather enough information in the field to be able to calculate the quantity involved.

10. If subgrade failures or slope failures are observed, the Production Management Division’s Office of Geotechnical Services should be contacted for further evaluation.

The above is not a full nor a complete list of items necessary to collect field information. Additional field research may be needed to accomplish the design.

Additional survey data may be required. If so, a survey may be performed to gather additional information such as structure inlet and outlet elevations, existing pavement grades, drainage areas, channel cross sections, horizontal or vertical realignment of existing facility, right-of-way needs, etc.

56-3.08 Plans Development

See Section 14-2.03 for these requirements.
56-3.09 Field Check

See Sections 14-2.03(03) and 14-2.03(08) for these requirements.

56-3.10 Pavement Design

See Section 52-9.0 for these requirements.

56-3.11 Revised Project Schedule

The designer should prepare a revised schedule for processing the project through the design phases including additional activities that were not included in the preliminary schedule. This revised schedule should be submitted to the district Office of Design and should be updated on a monthly basis or as required. The revised schedule should allow a minimum of 14 weeks prior to the letting date as the time at which the contract documents should be complete and ready for transmittal.

56-4.0 GENERAL DESIGN PARAMETERS

56-4.01 General Standards Requirements

All INDOT Standard Specifications and Standard Drawings will apply. All deviations from the Standard Specifications and Standard Drawings will be subject to approval by the Contract Administration Division director. A deviation from the Standard Specifications will require detail drawings and special provisions subject to the approval of the Contract Administration Division director. The designer should see Section 19-2.0 for instructions on writing special provisions.

56-4.02 References and Research Sources

References and research sources available for use as design references for supplemental information include the following:
56-4.03 Desirable and Minimum Pavement-Width Requirements

The values shown in Figures 56-4A, 56-4B, 56-4C, and 56-4D should be used for the design of pavement, travel-lane, shoulder, and curb-offset widths.

The figures are titled as follows:

56-4A    Pavement Width for Rural Two-Lane Road with Shoulders
56-4B    Pavement Width for Rural Road of 4 or More Lanes with Shoulders
56-4C    Pavement Width for Urban Two-Lane Road with Curbs
56-4D    Pavement Width for Urban Road of 4 or More Lanes with Curbs.

If an existing width is greater than the value shown in the figures, the existing width should be used.

The minimum width of pavement widening, where used, should be 2 ft for constructability. The maximum width of pavement widening should not exceed that shown in Section 52-9.02(09). If widening varies from side to side of the existing pavement, a strip map or a typical cross section showing widening by stations should be provided. If cut or fill slopes are required, cross sections should be provided.

56-4.04 Mainline-Pavement Considerations

Considerations to be made regarding specific mainline pavement and approaches items for each type of partial 3R treatment are shown in Figure 56-4E. Some of these items are further detailed below.
Work of a larger magnitude than that shown in Figure 56-4E for a given treatment may be done. Such work should be considered as a spot improvement, designed to the appropriate standards.

56-4.04(01) Auxiliary Lane

Incorporating or upgrading a turn lane, parking lane, passing blister, or other auxiliary lane to reduce the disruption of the flow of traffic should only be considered for a structural treatment. A geotechnical evaluation may be required. A partial 3R project involves few agreements and should require no additional right of way. The guidelines in Chapter 46 may not be attainable due to budgetary constraints and right-of-way acquisition. An auxiliary lane which cannot be considered in the project may be separately programmed as a spot improvement or into a future full 3R or 4R project. See Chapter 54 or 55 for appropriate requirements.

56-4.04(02) Castings

Castings need not be reset if the overlay depth is equal to the milling depth. However, if the finished grade is different from the original grade, the adjustment of the castings should be incorporated into the work.

In an unincorporated area, Department storm-sewer or sanitary-sewer castings should be adjusted to grade as required. In an incorporated area, the local utilities should be required to adjust castings as required. See Chapter 36 for more information.

In an area to be surface milled, all utility castings, and storm-sewer and sanitary-sewer castings, should be located and identified.

56-4.04(03) Cross Slopes

1. Travel Lanes. Pavement cross slopes on a tangent section should be reviewed for each type of partial 3R treatment. Improving pavement cross slope, where required, may be completed through staged construction, e.g., combining surface milling with pavement core investigation with a variable-depth cross section of HMA Intermediate course in accordance with the INDOT Standard Specifications prior to placing a uniform-depth HMA Surface course.
A preventative-maintenance treatment is exempt from crown correction only if an existing rural-pavement cross slope is 2%, or if an existing urban-pavement cross slope is 1.5 to 3%. If the slope is outside this range, a combination of surface milling and a uniform-depth HMA Surface course should be used.

2. **Shoulders.** For a paved shoulder of 4 ft or narrower, the cross slope should match the mainline cross slope. For a paved shoulder wider than 4 ft, the cross slope should match the existing shoulder slope, or should desirably be 4%. An aggregate- or earth-shoulder slope should be 4% to 8%. In a horizontal curve, shoulder slope should be determined in accordance with Section 43-3.0.

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**56-4.04(04) Curbs**

In areas where the curb height is not adequate for drainage, the pavement adjacent to the curb should be milled to the depth required for adequate drainage. If the curb is not structurally adequate, curb replacement should be considered. The pavement in such an area should be evaluated for possible future replacement.

**56-4.04(05) Monuments**

All existing Department monuments should be perpetuated. The designer is responsible for contacting the county surveyor for a list of monuments to be reset, witnessed, and monumented. Each affected monument is to be shown on the plans, or the required information is to be provided prior to construction.

**56-4.04(06) Sight-Distance Improvement**

Existing geometrics should be maintained if no adverse accident history exists. See Chapter 55 for desirable geometric criteria.

**56-4.04(07) Subsurface Drainage**

Subsurface drainage should be considered and perpetuated where it currently exists. For a structural treatment, addition of subsurface drainage should be considered. See Section 52-10.0 for subsurface-drainage-design requirements.
56-4.04(08) Superelevation and Horizontal Curve

For a functional or structural treatment, evaluation of an existing curve and superelevation should be performed. For a structural treatment, the pavement slope should be in accordance with the superelevation requirements shown in Section 43-3.0 where possible.

56-4.04(09) Surface Milling

Milling of HMA pavement will be used to adjust roadway cross section, develop or maintain curb exposure, remove wheel ruts, tie the new pavement into existing pavement, improve drainage, or remove undesirable areas or layers of pavement. Surface milling will be required as described in Section 52-7.05. Existing layers of HMA Surface Sand on or near the surface should be removed. Cores should be taken and analyzed by the district Office of Testing to ensure that the proposed milling can be performed. Where milling is proposed near a signalized intersection, the designer should coordinate with the district Office of Traffic to either avoid or replace existing signal loops. Details for milling at the project termini are shown in the INDOT Standard Drawings.

56-4.04(10) Urban Surface Drainage

Improvements to an urban surface-drainage system to correct water ponding that may be causing pavement stability problems may be included in a partial 3R project. Where surface milling is required to achieve drainage of a low location where water collects, or to remove existing asphalt, pavement cores should be obtained in the area to assess the pavement structure.

56-4.05 Approaches

It has been the practice of the Department to maintain the surfaces of the approaches to its routes. The limits and type of treatment vary with the type of approach. The treatments and limits used to maintain these approaches in conjunction with partial 3R work are provided herein.

Each approach should be in accordance with the INDOT Standard Specifications and Standard Drawings. Approach-data tables may be provided for supplemental information. See Chapters 46 and 52 for approach-design criteria where approach improvements are to be made.
56-4.05(01) Public-Road Approach

This type of approach should be overlaid to the apparent right-of-way line, unless the approach is another Department-maintained route which has recently been, or is scheduled to be treated within two years of completion of the partial 3R project. Shoulders should be constructed on each approach where shoulders exist or are being constructed on the mainline. The approach geometry should comply with the INDOT Standard Drawings as nearly as possible, especially where approaching a mainline pavement with AADT ≥ 3,000. An existing paved public-road approach should be overlaid to match the existing mainline’s pavement-edge elevation, and tapered to match the profile on the approach at the apparent right-of-way line through the use of a milled notch at the terminus. See the INDOT Standard Drawings for details.

56-4.05(02) Drive

1. **Asphalt.** The partial 3R treatment of an asphalt drive consists of a 3-ft wide wedge of HMA for Approaches placed adjacent to the mainline or shoulder pavement as shown on the INDOT Standard Drawings. This 3-ft width, depending on the depth of the mainline overlay, may not be practical and may need to be extended to prevent a hump or adverse rollover (grade break) that is unacceptable.

2. **Concrete.** For a concrete drive, a wedge of HMA for Approaches should be placed over the concrete terminating in a milled notch as shown on the INDOT Standard Drawings. The approach design length is based on the overlay depth on the mainline and an acceptable resultant grade on the approach.

3. **Aggregate.** For an aggregate drive adjacent to a nonpaved shoulder, a 3-ft widening with HMA for Approaches should be placed adjacent to the outer edge of the mainline or shoulder pavement. After placement of the widening, if a grade differential exists, it should be wedged out with compacted aggregate. Rollover criteria should be considered.

4. **Field Entrance.** This type of drive is earth. Fill is placed as required and compacted to the edge-of-shoulder or -pavement elevation.
56-4.05(03) Mailbox Approach

An existing mailbox approach may be substandard and most often cannot be corrected within the existing right of way. In a preventative maintenance or functional treatment, this type of approach should be overlaid to match the mainline elevation by use of the same paving material specified for the shoulder. In a structural treatment, a substandard approach deemed to be a hazard that can be improved should be improved to the geometrics shown in the INDOT Standard Drawings. If the standardized-approach limits intercept the mailbox location, the mailbox should be reset. If the shoulder is not to be paved, a mailbox approach should be provided as described in Section 52-9.02(08).

56-4.06 Roadside Considerations

The designer must keep focused on the objectives of the scope of work that has been established for an individual partial 3R project in order to apply the appropriate roadside-safety improvements.

Roadside-safety improvements should be considered as described in Figure 56-4F. Some of these are further detailed below.

56-4.06(01) Guardrail

Where required, a Guardrail Summary Table should be prepared for each area with guardrail placement or modification.

The field notes and design calculations should be submitted with the project file. Guardrail requiring modifications not shown in the INDOT Standard Drawings should be detailed on the plans.

A guardrail end treatment type I may be in place but is now inappropriate due to higher design-year average annual daily traffic counts than it was warranted for. Such treatment should be considered for replacement with a type OS or MS treatment as appropriate for a functional or structural treatment.
56-4.06(02) Linear Grading

Linear grading may be considered only where earth is wedged at the outside edge of the shoulder, the profile grade has been raised due to overlaying or widening the pavement, or earth is wedged behind guardrail to obtain the required earth backup for the posts.

56-4.06(03) Mailbox Assembly

Existing mailbox assemblies may remain in place during the performance of most partial 3R work. If a mailbox assembly is to be moved to accommodate a functional or structural treatment, an assembly as shown in the INDOT Standard Drawings should be considered for the replacement. See Section 49-3.01(02) for design criteria.

** ** PRACTICE POINTER ** **

If a mailbox height relative to the profile grade is lessened by overlaying its approach, and the box need not be replaced, its height should be adjusted accordingly.

56-4.06(04) Side Ditches

For a structural treatment, efforts should be made to re-establish drainage patterns and grades similar to the original construction. Where right of way is sufficient, efforts should be made to establish flow lines in accordance with Section 52-10.04(04) Item 2.

56-4.06(05) Side Slopes

For a preventative maintenance or a functional treatment, side slopes of steeper than 3:1 are acceptable.

A roadside slope which appears to be steeper than 3:1 in a structural treatment will require a survey preparation of shoulder cross-section to determine the slope. Each location which appears to be hazardous should be analyzed to determine if an adverse accident history exists, if it is cost effective to provide guardrail, or if a slope correction to a traversable level can be made.
Possible guardrail locations will be identified at the field check. See Sections 45-3.01 and 45-3.02 for guidance in determining side slope. Where significant widening is proposed on the side of an existing embankment, preliminary plans with cross sections should be sent to the Office of Geotechnical Services for evaluation.

56-4.06(06) Sidewalk [Rev. Feb. 2014]

This work may be incorporated into a partial 3R project as shown in Figure 56-4F. Partial 3R work deemed as an alteration must include curb ramps in the scope of work. See Section 51-1.04.

56-4.07 Culvert and Drainage-Structure Considerations

Culvert modification or replacement requirements for structural-treatment work are described in Section 31-4.04.

56-4.08 Traffic-Related Work

Traffic-related safety improvements should be considered as described in Figure 56-4F. Some of these items are further discussed below.

56-4.08(01) Highway Signs

Existing regulatory and warning signs anticipated to be impacted by structural-treatment construction operations should be reset or replaced as required in accordance with the INDOT Standard Specifications and Standard Drawings. See Section 502-1.0 for guidelines regarding highway signs.

A summary sheet or details should be included in the plans to list the locations for new and replacement sign types and required sign posts sizes and quantities.
56-4.08(02) Pavement Markings and Delineation

1. Markings. All permanent pavement markings, including transverse markings, should be replaced in kind. The district Office of Traffic should review the locations and quantities for such markings. The designer should contact the district Office of Traffic to coordinate the desired pavement-marking plan. New locations for markings should not be included in the project unless approved by the district Office of Traffic. The designer should consider the use of pavement markings as described in Section 502-2.0.

2. Snowplovable Raised Pavement Markers (RPMs). The designer should contact the district Office of Traffic to confirm the existence of RPMs within the project limits and for layout patterns that deviate from the INDOT Standard Drawings. See Section 502-2.02(12) for design criteria, and the INDOT Standard Drawings for basic layouts. If no existing RPMs are present, placement of new ones should be considered for a functional or structural treatment in accordance with Department policy.

All existing RPMs should be reviewed for replacement. Where RPMs exist, the designer has the following options for replacing removed RPMs.

a. Install refurbished castings and new prismatic reflectors.

b. Install new castings and new prismatic reflectors.

c. Replacements will be programmed by the district into the INSTIP annual replacement contract.

The first option is the most desirable and the third option is the most economical. A detailed plan sheet should be provided for each layout that differs from those shown on the INDOT Standard Drawings. A sheet may be included in the plans to list the color combinations and quantities of RPMs required.

56-4.08(03) Traffic Signal

Each detector-loop location should be identified and shown on the plans. Each detector housing affected by the overlay operation should be adjusted to grade. Adjustments to existing signal equipment such as signal-head reorientation, if required, may be incorporated into the work. A summary sheet or details should be included in the plans to list or detail the locations where loops, detector housings, or hand holes are to be replaced or adjusted.
A traffic signal should otherwise only be considered for upgrading or placement in a structural-treatment project.

56-4.09  Design Exception Criteria

56-4.09(01) Level One Criteria Subject to Design Exception [Rev. Feb. 2014]

If a work item is shown in Figure 56-4E or Figure 56-4F as A for a given type of treatment, a Level One or Level Two design exception request is required. A Level One exception is subject to approval of the Production Management Division director. Such work items are listed below.

1. Preventative Maintenance Treatment, A
   a. Sidewalk curb ramp, place in existing sidewalk or retrofit existing per ADA requirements

2. Functional Treatment, A.
   a. Cross-slope correction to 2%
   b. Sidewalk curb ramp, place in existing sidewalk or retrofit existing per ADA requirements

3. Structural Treatment, A.
   a. Cross-slope, convert tilt section to crown section
   b. Cross-slope correction to 2%
   c. Sidewalk curb ramp, place in existing sidewalk or retrofit existing per ADA requirements
   d. Superelevation rate, improve to standard

56-4.09(02) Level One Criteria Not Subject to Design Exception [Rev. Feb. 2014]

Some work items shown in Figure 56-4E or Figure 56-4F as B or C for a given type of treatment are Level One criteria, but a design exception request is not required. Such B work items are listed below.

1. Preventative Maintenance Treatment, B.
   a. Cross-slope correction to 2%
2. **Functional Treatment, B.**
   a. Cross-slope, convert tilt section to crown section
   b. Shoulder width
   c. Superelevation rate, improve to standard

3. **Structural Treatment, B.**
   a. Lane width
   b. Shoulder width

The C work items are listed below.

4. **Preventative Maintenance Treatment, C.**
   a. Bridge railing, upgrade to current standards
   b. Cross-slope, convert tilt section to crown section
   c. Lane width
   d. Shoulder width
   e. Superelevation rate, improve to standard

5. **Functional Treatment, C.**
   a. Bridge railing, upgrade to current standards
   b. Lane width

6. **Structural Treatment, B.**
   a. Bridge railing, upgrade to current standards

**56-4.10 Maintenance of Traffic**

A partial 3R project should be able to be completed without a road closure. If a road closure is necessary, the designer should follow the procedure described in Sections 82-2.0 and 82-7.02.

The designer should ascertain that there is sufficient roadway and shoulder width to safely accommodate both the contractor’s equipment and the flow of traffic. If a roadway shoulder is to be utilized to carry traffic during construction, it must be capable of withstanding the expected traffic load and volume. A traffic-control plan should be developed as described in Section 82-2.0. The designer should consider the use of temporary traffic-control devices as described in Chapter 83.
### STATIONING MARKING OPTIONS

<table>
<thead>
<tr>
<th>OPTION 1</th>
<th>OPTION 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Place a mark every 50 ft</td>
<td>Place a mark every 25 ft</td>
</tr>
<tr>
<td>Place an abbreviated station number every 100 ft</td>
<td>Place an abbreviated station number every 100 ft</td>
</tr>
<tr>
<td>Place a complete station number every 1000 ft</td>
<td>Place a complete station number every 1000 ft</td>
</tr>
</tbody>
</table>

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### STATIONING CONVENTIONS

Figure 56-3A
## COLLECTION OF FIELD DATA

**Figure 56-3B**

<table>
<thead>
<tr>
<th>Roadway/Street</th>
<th>Commercial Drives</th>
<th>Private Drives</th>
<th>Mailboxes</th>
<th>Wedge</th>
<th>Patch</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>U</td>
<td>I</td>
<td>U</td>
<td>I</td>
</tr>
<tr>
<td>AADT</td>
<td>Total Paved Width</td>
<td>Travel-Lane Width</td>
<td>Paved-Shoulder Width</td>
<td>Sealed-Agg. Shld. Width</td>
<td></td>
</tr>
<tr>
<td>------------</td>
<td>-------------------</td>
<td>-------------------</td>
<td>----------------------</td>
<td>-------------------------</td>
<td></td>
</tr>
<tr>
<td>≥ 5,000</td>
<td>D = 28 ft</td>
<td>D = 12 ft</td>
<td>D = 6 ft</td>
<td>N / A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M = 26 ft</td>
<td>M = 12 ft</td>
<td>M = 1 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3,000 ≤ AADT &lt; 5,000</td>
<td>D = 28 ft</td>
<td>D = 12 ft</td>
<td>D = 4 ft</td>
<td>D = 3 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M = 26 ft</td>
<td>M = 12 ft</td>
<td>M = 1 ft</td>
<td>M = 2 ft</td>
<td></td>
</tr>
<tr>
<td>400 ≤ AADT &lt; 3,000</td>
<td>D = 28 ft</td>
<td>D = 12 ft</td>
<td>D = 2 ft</td>
<td>D = 3 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M = 24 ft</td>
<td>M = 11 ft</td>
<td>M = 1 ft</td>
<td>M = 2 ft</td>
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</tr>
<tr>
<td>AADT &lt; 400</td>
<td>D = 26 ft</td>
<td>D = 11 ft</td>
<td>D = 2 ft</td>
<td>D = 2 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M = 22 ft</td>
<td>M = 10 ft</td>
<td>M = 1 ft</td>
<td>M = 1 ft</td>
<td></td>
</tr>
</tbody>
</table>

D = Desirable, M = Minimum.

1 Includes widths of travel lanes and paved shoulders only.
2 Should be widened on a curve where possible as described in Section 43-2.06.
3 For a road with a TWLTL, the total width will be increased by the applicable width of a TWLTL.

**PAVEMENT WIDTH FOR RURAL 2-LANE ROAD WITH SHOULDERS**

*Figure 56-4A*
<table>
<thead>
<tr>
<th></th>
<th>4 Lanes, Total Paved Width</th>
<th>6 Lanes, Total Paved Width</th>
<th>Travel-Lane Width</th>
<th>Outside Paved-Shoulder Width</th>
<th>Median Paved-Shoulder Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Divided</td>
<td>D = 36 ft, M = 29 ft</td>
<td>D = 48 ft, M = 40 ft</td>
<td>D = 12 ft, M = 11 ft</td>
<td>D = 8 ft, M = 4 ft</td>
<td>D = 4 ft, M = 3 ft</td>
</tr>
<tr>
<td>Undivided</td>
<td>D = 32 ft, M = 26 ft</td>
<td>D = 44 ft, M = 37 ft</td>
<td>D = 12 ft, M = 11 ft</td>
<td>D = 8 ft, M = 4 ft</td>
<td>N / A</td>
</tr>
</tbody>
</table>

D = Desirable, M = Minimum.

1 Includes widths of travel lanes and paved shoulders for one direction of travel on a divided road.

2 For a road with a TWLTL, the total width will be increased by the applicable width of a TWLTL.

**PAVEMENT WIDTH FOR RURAL ROAD OF 4 OR MORE LANES WITH SHOULDERS**

Figure 56-4B
<table>
<thead>
<tr>
<th></th>
<th>Total Width</th>
<th>Travel-Lane Width</th>
<th>TWLTL</th>
<th>Curb-Offset Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1,2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suburban</td>
<td>D = 26 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 1 ft</td>
</tr>
<tr>
<td></td>
<td>M = 20 ft</td>
<td>M = 10 ft</td>
<td>M = 12 ft</td>
<td>M = 0 ft</td>
</tr>
<tr>
<td>Intermediate</td>
<td>D = 26 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 1 ft</td>
</tr>
<tr>
<td></td>
<td>M = 22 ft</td>
<td>M = 10 ft</td>
<td>M = 11 ft</td>
<td>M = 0 ft</td>
</tr>
<tr>
<td>Built-Up</td>
<td>D = 26 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 1 ft</td>
</tr>
<tr>
<td></td>
<td>M = 20 ft</td>
<td>M = 10 ft</td>
<td>M = 10 ft</td>
<td>M = 0 ft</td>
</tr>
</tbody>
</table>

D = Desirable, M = Minimum. TWLTL = two way left turn lane.

1 Total width face to face of curb.
2 For a road with a TWLTL, the total width will be increased by the applicable width of a TWLTL.
3 See Section 40-1.02 for definitions.

**PAVEMENT WIDTH FOR URBAN 2-LANE ROAD WITH CURBS**

*Figure 56-4C*
<table>
<thead>
<tr>
<th></th>
<th>4 Lanes, Total Width</th>
<th>6 Lanes, Total Width</th>
<th>Travel-Lane Width</th>
<th>TWLTL</th>
<th>Curb-Offset Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suburban</td>
<td>D = 28 ft</td>
<td>D = 40 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 2 ft</td>
</tr>
<tr>
<td></td>
<td>M = 24 ft</td>
<td>M = 35 ft</td>
<td>M = 11 ft</td>
<td>M = 11 ft</td>
<td>M = 1 ft</td>
</tr>
<tr>
<td>Intermediate</td>
<td>D = 28 ft</td>
<td>D = 40 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 2 ft</td>
</tr>
<tr>
<td></td>
<td>M = 24 ft</td>
<td>M = 35 ft</td>
<td>M = 11 ft</td>
<td>M = 11 ft</td>
<td>M = 1 ft</td>
</tr>
<tr>
<td>Built-Up</td>
<td>D = 28 ft</td>
<td>D = 40 ft</td>
<td>D = 12 ft</td>
<td>D = 12 ft</td>
<td>D = 2 ft</td>
</tr>
<tr>
<td></td>
<td>M = 22 ft</td>
<td>M = 32 ft</td>
<td>M = 10 ft</td>
<td>M = 11 ft</td>
<td>M = 1 ft</td>
</tr>
</tbody>
</table>

D = Desirable, M = Minimum. TWLTL = two way left turn lane.

1 Total width face to face of curb for one direction of a divided road.
2 For a road with a TWLTL, the total width will be increased by the applicable desirable or minimum width of a travel lane.
3 See Section 40-1.02 for definitions.

PAVEMENT WIDTH FOR URBAN ROAD OF 4 OR MORE LANES WITH CURBS

Figure 56-4D
### Pavement Treatment → Prvnt. Maint. Functional Structural

#### Approach,
- Drive, Relocate or Redesign
  - C C B
- Drive, wedge 3 ft adjacent to mainl. pvmt.
  - A A A
- Mailbox, Improve to Standard or Incorporate
  - C C B
- Mailbox, Overlay Existing
  - A A A
- Public Road, Treat to Mainline R/W Line
  - A A A

#### Auxiliary Lane,
- Improve to Current Standards or Incorporate
  - Channelization Lane
    - C C B
  - Climbing Lane
    - C C C
  - Parking Lane
    - C C B
  - Passing Blister
    - C C B
  - Turn Lane
    - C C B

#### Casting,
- Adjust to Grade
  - C B A

#### Cross-Slope Correction,
- Convert Tilt Section to Crown Section
  - C B A
- Correct to 2%
  - B A A

#### Curbs,
- Repair
  - B B B
- Replace
  - C B B

#### Drainage Structure,
- Repair, Clean, or Adjust
  - B B B

#### Intersection,
- Improve Sight Distance and Radii
  - C C B

#### Lanes,
- Widen to Minimum or Desirable Standards
  - C C B

#### Median,
- Convert to Two-Way Left-Turn Lane
  - C C B

#### Monument,
- Perpetuate
  - A A A

---

Key to letters A, B, and C is shown at the end of the table.

**PARTIAL 3R WORK**
Mainline Pavement and Approach Considerations

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<table>
<thead>
<tr>
<th>Pavement Treatment →</th>
<th>Prvnt. Maint.</th>
<th>Functional</th>
<th>Structural</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Patching</strong></td>
<td>B</td>
<td>A</td>
<td>A</td>
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<tr>
<td><strong>Shoulders,</strong></td>
<td>C</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Widen to Minimum or</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Desirable Standards</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sight-Distance Improvement,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal</td>
<td>C</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Vertical</td>
<td>C</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td><strong>Subsurface Drainage,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Underdrain, Clean or</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Repair</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Underdrain, Place or</td>
<td>C</td>
<td>C</td>
<td>B</td>
</tr>
<tr>
<td>Replace</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Superelevation Rate,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Improve to Current</td>
<td>C</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>Standards</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Underpass Vertical Clearance,</strong></td>
<td></td>
<td></td>
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<tr>
<td>Maintain Current</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Distance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Urban Surface Drainage,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mill as Required to</td>
<td>B</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Maintain or Correct</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Key to letters in table:

A = Item should be included as part of the project.
B = Item may be included.
C = Item should not be included. If it is considered, it should be programmed separately as a spot improvement.

Notes:

1 Minimum and desirable standards are shown in Figures 56-4A, 56-4B, 56-4C, and 56-4D.
2 Minimum and desirable standards are shown in Figures 56-4A, 56-4B, 56-4C, and 56-4D.
   Paved shoulders are desirable they can be placed without affecting drainage or side ditches.
3 Current standards are shown in Section 43-3.0
<table>
<thead>
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<th>Pavement Treatment →</th>
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<th>Structural</th>
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<tr>
<td><strong>Culvert,</strong></td>
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<tr>
<td>Extend</td>
<td>B</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>Modify</td>
<td>B</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>Place New</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>Repair and Clean</td>
<td>B</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>Replace</td>
<td>B</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>Headwalls, Remove</td>
<td>C</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td><strong>Eroded Area,</strong></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Grade and Seed or Sod</td>
<td>B</td>
<td>B</td>
<td>A</td>
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<tr>
<td><strong>Guardrail End Treatment,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Repair Damaged</td>
<td>A</td>
<td>A</td>
<td>A</td>
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<tr>
<td>Replace product not on appvd. list with appvd. prod.</td>
<td>B</td>
<td>B</td>
<td>B</td>
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<tr>
<td>Replace type I with type MS or OS as required.</td>
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<tr>
<td><strong>Highway Sign,</strong></td>
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<td></td>
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<tr>
<td>Replace</td>
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<td>C</td>
<td>B</td>
</tr>
<tr>
<td><strong>Impact Attenuator,</strong></td>
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<td></td>
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</tr>
<tr>
<td>Repair Damaged</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Replace product not on appvd. list with appvd. prod.</td>
<td>B</td>
<td>B</td>
<td>B</td>
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<tr>
<td><strong>Linear Grading</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>C</td>
<td>B</td>
</tr>
<tr>
<td><strong>Mailbox,</strong></td>
<td></td>
<td></td>
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<tr>
<td>Adjust Mounting Height Where Required</td>
<td>A</td>
<td>A</td>
<td>A</td>
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<tr>
<td>Replace Where Required</td>
<td>B</td>
<td>B</td>
<td>A</td>
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<td><strong>Obstruction-Free-Zone Clearance,</strong></td>
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<td></td>
<td></td>
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<tr>
<td>Remove Fixed Object &gt; 4 in. Above Ground</td>
<td>C</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td><strong>Pavement Markings and Delineation,</strong></td>
<td></td>
<td></td>
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<tr>
<td>Pavement Markings, Place</td>
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<td>A</td>
<td>A</td>
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<tr>
<td>Roadside Delineators, Place or Replace</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Raised Pavement Markers, Place</td>
<td>C</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Raised Pavement Markers, Replace</td>
<td>B</td>
<td>B</td>
<td>B</td>
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<tr>
<td><strong>Side Ditch,</strong></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Reshape or Riprap</td>
<td>B</td>
<td>B</td>
<td>B</td>
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Key to work incidental to paving is shown at the end of the table.

**PARTIAL 3R WORK**
Roadside, Culvert, and Traffic Considerations

**Figure 56-4F**
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<table>
<thead>
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<td><strong>Side Slope,</strong></td>
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<tr>
<td>Flatten to Traversable Level</td>
<td>C</td>
<td>C</td>
<td>B</td>
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<tr>
<td><strong>Sidewalk,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Repair or Replace <strong>per ADA requirements</strong></td>
<td>B</td>
<td>B</td>
<td>B</td>
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<tr>
<td><strong>Sidewalk Curb Ramp at Intersection,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upgrade existing to ADA requirements</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Place in exist. sdwk. per ADA requirements</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td><strong>Traffic Barrier,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridge Railing, Upgrade to Current Standards</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>Guardrail, Repair or Replace Damaged</td>
<td>A</td>
<td>A</td>
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<tr>
<td>Guardrail, Replace Obsolete ¹ or Weathered</td>
<td>C</td>
<td>B</td>
<td>B</td>
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<tr>
<td>Guardrail, Place or Lengthen to Current Standards ²</td>
<td>C</td>
<td>B</td>
<td>B</td>
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<tr>
<td>Guardrail to Bridge Railing, Connect</td>
<td>C</td>
<td>A</td>
<td>A</td>
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<tr>
<td>Guardrail Transition, Upgrade to Current Standards</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td><strong>Traffic Signal,</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Add or Upgrade</td>
<td>C</td>
<td>C</td>
<td>B</td>
</tr>
<tr>
<td>Detector Loop or Handhole, Perpetuate</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

Key to work incidental to paving:

A = Item should be included as part of the project.
B = Item may be included.
C = Item should not be included. If it is considered, it should be programmed separately as a spot improvement.

Notes:

¹ Obsolete guardrail should be treated as shown in Section 49-4.02.
² Treat as described in Section 55-5.04.
³ For example, tree, bush, post, rock, private sign, etc. See Section 55-5.02 for obstruction-free-zone information.

PARTIAL 3R WORK
Roadside, Culvert, and Traffic Considerations

Figure 56-4F
(Page 2 of 2)
CHAPTER 303

Roadside Safety

<table>
<thead>
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<th>Design Memorandum</th>
<th>Revision Date</th>
<th>Sections Affected</th>
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<td>May 2013</td>
<td>49-5.01</td>
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<td>16-17</td>
<td>April 2016</td>
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</tbody>
</table>
NOTE: This chapter is currently being re-written and its content will be included in Chapter 303 in the future.

<table>
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<th>Revision Date</th>
<th>Sections Affected</th>
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<td>April 2013</td>
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<td>April 2016</td>
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</tbody>
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CHAPTER 49

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 53. Design speed for a 3R or partial 4R project on a freeway is provided in Chapter 54.

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 $\leq$ 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 \( \times \) 1.2 = 18 ft  

The transition length, \( L = 3.1 \times 55 = 171 \) ft

* * * * * * * * * *

**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. Cut Slope for New Facility. 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. Auxiliary Lane. 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 49-2I, Clear-Zone Application for Auxiliary Lane or Ramp, illustrates the clear-zone application for an auxiliary lane next to the mainline.
7. **Ramp.** 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 illustrates the clear-zone application for a ramp/mainline configuration.

* * * * * * * * * *

**Example 49-2.2**

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

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

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

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

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, $F_{w\text{run}}$, as follows:

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

Where:
- $W_f$ = Width of foreslope, 10 ft
- $W_d$ = Width of ditch, 4 ft
- $F_{\text{rise}}$ = Foreslope rise, 1
- $F_{\text{run}}$ = Foreslope run, 6
- $D_{\text{rise}}$ = Ditch slope rise, 1
- $D_{\text{run}}$ = Ditch slope run, since flat, use 10
\[ F_{\text{w/run}} = \frac{10 + 4}{10(1/6) + 4(1/10)} = \frac{14}{2.07} = 6.8 \]

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:

\[ CZ_{%FD} = \frac{100(W_s + W_f + W_d)}{CZ_F} \]  
(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:

\[ CZ_{BR} = \frac{CZ_B (100 - CZ_{%FD})}{100} \]  
(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:

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

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

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 $V \leq 35$ 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. Continuous 24-Hours Parking. 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. Parking Lane Used as a Travel Lane. 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

49-3.01 Roadside Hazards

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 ft²;

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 = $3,000 \times 4.0 = 12,000$

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, $4 \text{ ft} \leq \text{Width} \leq 8$ 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. **In Fill, Reconstruction Project.** 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. **In Fill, New Facility.** 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. **In Cut, Reconstruction Project.** 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.

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#### 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. **66-in. or Larger-Diameter Culvert**. 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. **Pipe in the Median.** 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. **Box Culvert or Three-Sided Structure.** See Figure 49-3I, 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. **12-in. to 60-in. Diameter Culvert in the Median.** 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. **12-in. Diameter Culvert.** 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. **15-in. to 60-in. Diameter Culvert in Side Ditch.** 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
   - = 25 ft x 1.4 = 35 ft
The curve correction factor is applied only to the outside (high side) of a horizontal curve.

2. **Shoulder-Pier Clearance.** 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. **Median Pier.** 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 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:
   \[
   \frac{35 - 11}{6} = 4 \text{ ft}^3
   \]
4. Elevation differential from slope break for 6:1 cut slope at 35 ft is as follows:
   \[
   \frac{35 - 11}{6} = 4 \text{ ft}^4
   \]
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 ft² 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 [Rev. Apr. 2016]**

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. See Section 49-6.04 for slope considerations in front of median barrier.

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 practicability 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 [Rev. Apr. 2016]

The adjacent shoulder slope or embankment slope in front of a semi-rigid roadside barrier (e.g. strong post w-beam or thrie-beam) should desirably be 10:1 or flatter. New installations of these barriers on slopes 6:1 or steeper is not recommended. Where site conditions dictate, a 6:1 slope, may be provided in front of the barrier, but due to the trajectory of a vehicle bumper the barrier should be placed at least 12 ft beyond the shoulder break point. The 2011 AASHTO Roadside Design Guide section 5.6.2 discusses placement considerations with regard to slopes and the presence of curbs.

Slopes for barriers placed in the median are discussed in section 49-6.04(01).

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:

\[ L_{TOT} = L_{ADV} + L_{HAZ} + L_{OPP} \]  

[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. **Graphical Solution, Tangent or Inside Horizontal Curve.** 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. **Graphical Solution, Outside Horizontal Curve.** 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. **Mathematical Solution, Tangent Section Only.** The required length of need may be calculated using the formulas as follows:

   For a flared barrier installation:

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

   \[
   Y = L_H \cdot \frac{L_H}{(L_R)} (X) \tag{Equation 49-4.3}
   \]
For a parallel barrier installation:

\[ X = \frac{L_R \left( L_H - L_2 \right)}{L_H} \]  

[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(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 \( LET \) 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.
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. **Support Located ≤ 16 ft from Edge of Travel Lane.** 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. **Support Located > 16 ft from the Edge of Travel Lane.** 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**

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**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, \( CZ = 46 \text{ ft} \).

2. From Figure 49-4E, Design Elements for Barrier Length of Need, determine runout length, \( L_R = 460 \text{ ft} \).

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).

* * * * * * *

Example 49-4.3

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 [Rev. May 2013]

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 desirable distance from the face of guardrail to the shoulder break point is 3'-5" ft. This provides a 2-ft offset from the back of the guardrail post. In a restrictive situation, the offset from the back of the guardrail post may be 0 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. High-Tension Cable-Barrier System (CBS). 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 [Rev. Apr. 2011]

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. **Warrants.** 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. **Design Considerations.**

   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. For the desirable longest-run length, see Opening in CBS Run for Law-enforcement or Emergency-Response Vehicle.

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 minimum lateral clearance to a rigid obstacle such as a bridge pier, 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 transitioned horizontally at a taper rate of 50:1 or flatter. 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 at a taper rate of 50:1 or flatter.

j. CBS at Official Median Crossover. For a CBS run termination at such a crossover, the CBS end terminal (end anchor) should be located preferably 3 to 5 ft beyond the points where the crossover radii are tangent with the edges of the adjacent travel lanes.

k. Changing Offset of CBS Run in a Median from Being Closer to One Roadway to Being Closer to the Opposing-Traffic Roadway. If a CBS run requires a change of lateral offset, the end anchors of the CBS run should be overlapped for the minimum distance between the anchors in each direction as described below. The minimum overlap 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.

m. Opening in CBS Run for Law-Enforcement or Emergency-Response Vehicle. The desirable longest run should be 6,000 ft to allow for a vehicle crossover, although an ultimate maximum of 10,000 ft is allowed under certain site conditions. The locations of such openings should be shown on the plans. The soil-stabilization method, if required, should be specified. Delineator posts with reflectors should be placed in a row to indicate that a crossover is available at the opening.
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. \( W \leq 24 \) and \( C < 4 \). Use nested guardrail including a 25-ft span over the structure as shown on the INDOT Standard Drawings.

2. \( 24 < W \leq 60 \) and \( 1.5 \leq C < 4 \). 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. \( W \) Not Limited and \( 4 \leq C < 5 \). 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. \( W \) Not Limited and \( C \geq 5 \). 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.

   \[
   \text{Adjusted construction-year AADT} = \frac{(\text{construction-year AADT shown on plans})(K_g)(K_c)(K_s)}{1000}
   \]
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, \( L_2 = 11 \text{ ft} \):

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, \( K_s = 0.7 \).

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

From Figure 49-6D(65, 70), Median-Barrier or Bridge-Railing Test-Level Selection, Design Speed 70 mph, for % Trk 10 ≤ % < 15, 7 < \( L_2 \) ≤ 12, and highway type as Divided, the appropriate \( T \) range is 2.6 < \( T \) ≤ 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, \( L_2 = 11 \text{ ft} \):

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(65, 70), for % Trk 10 ≤ % < 15, 7 < \( L_2 \) ≤ 12, and highway type as divided, the appropriate \( T \) range is 2.6 < \( T \) ≤ 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.

* * * * * * *
49-6.04 Median-Barrier Design

49-6.04(01) Median Slopes [Rev. Apr. 2016]

The slope in front of a median barrier should be 20:1 or flatter. Median slopes can affect the performance of the barrier. 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.

The 2011 AASHTO Roadside Design Guide, section 6.6 discusses barrier placement recommendations with regard to the effects of terrain and fixed objects within 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. **Type MS – Median Shoulder.** 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. **Type II.** 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. **Slopes.** All slopes in the area of a GRET should be graded in accordance with the INDOT Standard Drawings.

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

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

4. **Opening Near a Bridge.** 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.

**49-8.03 Bridge-Railing Transitions**

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. **Type TGS-1 – Transition, Guardrail, Side-mounted, 1 tube.** 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. **W1.** This attenuator width is required for an obstruction that is not more than 3 ft wide.
2. **W2.** 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. **W3.** 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 53, 54, or 55, 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. **Function Key 1.** 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. **Function Key 2.** This key will store the problem variables and basic input data.

3. **Function Key 3.** 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. **Function Key 4.** This key will let the user access the HELP menu which contains detailed information on every aspect of ROADSIDE.

5. **Function Key 5.** This key will display, and allow the user to change, the basic input (global) values.
6. **Function Key 6.** This key will display the relationship between severity index and cost as derived from the accident costs included in the basic input values.

7. **Function Key 7.** This key will list all file names on the ROADSIDE disk.

8. **Function Key 8.** This key lists the percentage of accident types included for each severity index value.

9. **Function Key 9.** 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. **Function Key 10.** This key is used to exit the program. No data are stored via this function. Data should be stored using Function Key 2.

### 49-10.02 Basic Input Data

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**

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td><strong>Traffic Volume</strong></td>
<td>two-way ADT</td>
</tr>
<tr>
<td></td>
<td><strong>Growth Rate</strong></td>
<td>percent</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Roadway Type</td>
<td>undivided (U), divided (D), one-way (O)</td>
</tr>
<tr>
<td></td>
<td>Lanes of Adjacent Traffic</td>
<td>number of lanes</td>
</tr>
<tr>
<td></td>
<td>Width of Each Lane</td>
<td>feet</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Roadway Curvature Adjustment</td>
<td>degrees</td>
</tr>
<tr>
<td></td>
<td>Roadway Grade Adjustment</td>
<td>percent</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Design Speed</td>
<td>miles per hour</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Hazard Offset from Driving Lane</td>
<td>A, feet</td>
</tr>
<tr>
<td></td>
<td>Hazard Length (parallel to road)</td>
<td>L, feet</td>
</tr>
<tr>
<td></td>
<td>Hazard Width (perpendicular to road)</td>
<td>W, feet</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>Severity Index for:</td>
</tr>
<tr>
<td></td>
<td>upstream side of hazard (SU)</td>
</tr>
<tr>
<td></td>
<td>downstream side of hazard (SD)</td>
</tr>
<tr>
<td></td>
<td>upstream corner of hazard (CU)</td>
</tr>
<tr>
<td></td>
<td>downstream corner of hazard (CD)</td>
</tr>
<tr>
<td></td>
<td>longitudinal face of hazard (FACE)</td>
</tr>
</tbody>
</table>

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 49-10P 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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Project Life</td>
<td>years</td>
</tr>
<tr>
<td></td>
<td>Discount Rate</td>
<td>percent</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Installation Cost</td>
<td>dollars</td>
</tr>
<tr>
<td>12</td>
<td>Repair Cost (per accident)</td>
<td>dollars</td>
</tr>
<tr>
<td>13</td>
<td>Routine Maintenance Cost (per year)</td>
<td>dollars</td>
</tr>
<tr>
<td>14</td>
<td>Salvage Value</td>
<td>dollars</td>
</tr>
</tbody>
</table>
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.

* * * * * * * * * *

**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. **Accident Cost.** Annualized cost using *RDG* accident cost default values.

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$2,060</td>
<td>$0</td>
<td>$2,060</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$858</td>
<td>$392</td>
<td>$1,250</td>
<td>3.1</td>
</tr>
<tr>
<td>Option 3</td>
<td>$225</td>
<td>$625</td>
<td>$850</td>
<td>2.9</td>
</tr>
<tr>
<td>Option 4</td>
<td>$591</td>
<td>$441</td>
<td>$1,032</td>
<td>3.3</td>
</tr>
</tbody>
</table>
Annualized Cost for FHWA T 7570.1 accident cost values.

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$4,966</td>
<td>$0</td>
<td>$4,966</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$1,661</td>
<td>$392</td>
<td>$2,053</td>
<td>8.4</td>
</tr>
<tr>
<td>Option 3</td>
<td>$542</td>
<td>$625</td>
<td>$1,167</td>
<td>7.1</td>
</tr>
<tr>
<td>Option 4</td>
<td>$1,240</td>
<td>$441</td>
<td>$1,681</td>
<td>8.4</td>
</tr>
</tbody>
</table>

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).

<table>
<thead>
<tr>
<th>Impact Location</th>
<th>Option 1</th>
<th>Option 2</th>
<th>Option 3</th>
<th>Option 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream side</td>
<td>5.5/5.4</td>
<td>3.0/3.4</td>
<td>5.5/5.4</td>
<td>4.0/3.2</td>
</tr>
<tr>
<td>Downstream side</td>
<td>5.5/5.4</td>
<td>3.0/3.4</td>
<td>5.5/5.4</td>
<td>4.0/3.2</td>
</tr>
<tr>
<td>Upstream corner</td>
<td>6.0/5.5</td>
<td>3.0/3.4</td>
<td>6.0/5.4</td>
<td>4.0/3.2</td>
</tr>
<tr>
<td>Downstream corner</td>
<td>6.0/5.5</td>
<td>3.0/3.4</td>
<td>6.0/5.4</td>
<td>4.0/3.2</td>
</tr>
<tr>
<td>Face</td>
<td>4.8/4.2</td>
<td>2.7/3.2</td>
<td>4.8/4.2</td>
<td>4.0/3.2</td>
</tr>
</tbody>
</table>
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).

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$1,629</td>
<td>$0</td>
<td>$1,629</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$1,395</td>
<td>$392</td>
<td>$1,787</td>
<td>0.6</td>
</tr>
<tr>
<td>Option 3</td>
<td>$167</td>
<td>$625</td>
<td>$792</td>
<td>2.3</td>
</tr>
<tr>
<td>Option 4</td>
<td>$310</td>
<td>$441</td>
<td>$751</td>
<td>3.0</td>
</tr>
</tbody>
</table>

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:

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

Summary:

Annualized cost without accommodating for lane distribution.

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

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

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$15,946</td>
<td>$0</td>
<td>$15,946</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$8,012</td>
<td>$1,528</td>
<td>$9,540</td>
<td>5.2</td>
</tr>
<tr>
<td>Option 3</td>
<td>$7,152</td>
<td>$1,050</td>
<td>$8,202</td>
<td>8.4</td>
</tr>
<tr>
<td>Option 4</td>
<td>$3,426</td>
<td>$3,576</td>
<td>$7,002</td>
<td>3.5</td>
</tr>
</tbody>
</table>
Annualized cost with lane distribution - adjusting with user factor.

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

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).

<table>
<thead>
<tr>
<th>Input Variable</th>
<th>Option 1</th>
<th>Option 2</th>
<th>Option 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral Distance (A)</td>
<td>35’</td>
<td>29’</td>
<td>34’</td>
</tr>
<tr>
<td>Long. Length (L)</td>
<td>50’</td>
<td>130’</td>
<td>210’</td>
</tr>
<tr>
<td>Width (W)</td>
<td>3’</td>
<td>15’</td>
<td>5’</td>
</tr>
<tr>
<td>Installation Cost</td>
<td>$0</td>
<td>$10,000</td>
<td>$7,000</td>
</tr>
<tr>
<td>Repair Cost</td>
<td>$0</td>
<td>$100/acc</td>
<td>$0</td>
</tr>
<tr>
<td>Maintenance Cost</td>
<td>$0</td>
<td>$20/year</td>
<td>$10/year</td>
</tr>
<tr>
<td>Severity Index (Face)</td>
<td>6.5</td>
<td>4.3</td>
<td>4.3</td>
</tr>
<tr>
<td>Severity Index (Side)</td>
<td>6.5</td>
<td>4.8</td>
<td>4.8</td>
</tr>
</tbody>
</table>
Summary: Annualized cost - FHWA T 7570.1 accident costs.

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$3,913</td>
<td>$0</td>
<td>$3,913</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$2,507</td>
<td>$759</td>
<td>$3,266</td>
<td>1.9</td>
</tr>
<tr>
<td>Option 3</td>
<td>$2,410</td>
<td>$525</td>
<td>$2,935</td>
<td>2.9</td>
</tr>
<tr>
<td>Option 4</td>
<td>$2,410</td>
<td>$422</td>
<td>$2,832</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Annualized cost - RDG accident costs.

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Accident Cost</th>
<th>Agency Cost</th>
<th>Total Cost</th>
<th>B/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1</td>
<td>$1,581</td>
<td>$0</td>
<td>$1,581</td>
<td>n/a</td>
</tr>
<tr>
<td>Option 2</td>
<td>$1,117</td>
<td>$759</td>
<td>$1,875</td>
<td>0.6</td>
</tr>
<tr>
<td>Option 3</td>
<td>$1,081</td>
<td>$525</td>
<td>$1,606</td>
<td>1.0</td>
</tr>
<tr>
<td>Option 4</td>
<td>$1,081</td>
<td>$422</td>
<td>$1,503</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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.
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:

<table>
<thead>
<tr>
<th>Design Speed</th>
<th>Cut Slopes</th>
<th>Fill Slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3:1</td>
<td>4:1</td>
</tr>
<tr>
<td>≤ 40</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>45-50</td>
<td>1.23</td>
<td>1.14</td>
</tr>
<tr>
<td>55</td>
<td>1.33</td>
<td>1.25</td>
</tr>
<tr>
<td>60</td>
<td>1.63</td>
<td>1.44</td>
</tr>
<tr>
<td>65-70</td>
<td>1.56</td>
<td>1.27</td>
</tr>
</tbody>
</table>

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 12 ft for AADT greater than 1500. The longitudinal length of the guardrail was calculated to be 1000 ft + 2*LR, where LR is from Figure 49-4E. The width of guardrail was assumed to be 2 ft.
   
   f. Line 9. The severity indices from Figures 49-10H and 49-10I 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. Embankment Calculations. 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 be 10 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.
<table>
<thead>
<tr>
<th>Design Speed Year AADT, $T$</th>
<th>Foreslopes</th>
<th>Backslopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>6:1 or Flatter</td>
<td>5:1 or 4:1</td>
<td>3:1</td>
</tr>
<tr>
<td>≤ 40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 750</td>
<td>7–10</td>
<td>7–10</td>
</tr>
<tr>
<td>750 ≤ $T$ &lt; 1500</td>
<td>10-12</td>
<td>12-14</td>
</tr>
<tr>
<td>1500 ≤ $T$ ≤ 6000</td>
<td>12-14</td>
<td>14-16</td>
</tr>
<tr>
<td>&gt; 6000</td>
<td>14-16</td>
<td>16-18</td>
</tr>
<tr>
<td>45 or 50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 750</td>
<td>10-12</td>
<td>12-14</td>
</tr>
<tr>
<td>750 ≤ $T$ &lt; 1500</td>
<td>12-14</td>
<td>16-20</td>
</tr>
<tr>
<td>1500 ≤ $T$ ≤ 6000</td>
<td>16-18</td>
<td>20-26</td>
</tr>
<tr>
<td>&gt; 6000</td>
<td>18-20</td>
<td>24-28</td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 750</td>
<td>12-14</td>
<td>14-18</td>
</tr>
<tr>
<td>750 ≤ $T$ &lt; 1500</td>
<td>16-18</td>
<td>20-24</td>
</tr>
<tr>
<td>1500 ≤ $T$ ≤ 6000</td>
<td>20-22</td>
<td>24-30</td>
</tr>
<tr>
<td>&gt; 6000</td>
<td>22-24</td>
<td>26-32*</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 750</td>
<td>16-18</td>
<td>20-24</td>
</tr>
<tr>
<td>750 ≤ $T$ &lt; 1500</td>
<td>20-24</td>
<td>26-32*</td>
</tr>
<tr>
<td>1500 ≤ $T$ ≤ 6000</td>
<td>26-30</td>
<td>32-40*</td>
</tr>
<tr>
<td>&gt; 6000</td>
<td>30-32*</td>
<td>36-44*</td>
</tr>
<tr>
<td>65 or 70</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 750</td>
<td>18-20</td>
<td>20-26</td>
</tr>
<tr>
<td>750 ≤ $T$ &lt; 1500</td>
<td>24-26</td>
<td>28-36*</td>
</tr>
<tr>
<td>1500 ≤ $T$ ≤ 6000</td>
<td>28-32*</td>
<td>34-42*</td>
</tr>
<tr>
<td>&gt; 6000</td>
<td>30-34*</td>
<td>38-46*</td>
</tr>
</tbody>
</table>

Notes:

* Where a site-specific investigation indicates a high probability of continuing crashes, or such occurrences are indicated by crash history, a clear-zone distance greater than that shown in the table may be provided. The clear-zone width may be limited to 30 ft for practicality and to provide a consistent roadway template if previous experience with similar projects or designs has indicated satisfactory performance.

1. For a foreslope of 3:1, recovery is less likely if it is unshielded. Fixed objects should not be present. Recovery of a high-speed vehicle that encroaches beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should take into consideration right of way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the through travel lane and the beginning of the 3:1 slope should influence the recovery area provided at the toe of slope.

CLEAR-ZONE WIDTH (ft)
FOR NEW CONSTRUCTION OR RECONSTRUCTION

Figure 49-2A
<table>
<thead>
<tr>
<th>RADIUS (ft)</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
<th>60</th>
<th>65</th>
<th>70</th>
</tr>
</thead>
<tbody>
<tr>
<td>2860</td>
<td>1.1</td>
<td>1.1</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td>2290</td>
<td>1.1</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>1910</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
</tr>
<tr>
<td>1640</td>
<td>1.1</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>1430</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
<td>1.4</td>
<td>--</td>
</tr>
<tr>
<td>1270</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
</tr>
<tr>
<td>1150</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>950</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>820</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>720</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>640</td>
<td>1.3</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>570</td>
<td>1.4</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>380</td>
<td>1.5</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

Notes:

1. Adjustments apply to the outside of a horizontal curve.
2. A curve with radius greater than 2860 ft does not require adjustments.
3. The applicable clear-zone distance is calculated as follows:
   \[ CZ_c = (K_{cz})(CZ_t) \]
   where: \( CZ_c \) = clear zone on curve, ft
   \( K_{cz} \) = curve correction factor
   \( CZ_t \) = clear zone on tangent section from Figure 49-2A, ft
4. For a curve radius not shown in the table, use a straight-line interpolation.
5. See Figure 49-2C for the application of \( CZ_c \) to the roadside around a curve.

CLEAR-ZONE ADJUSTMENT FACTOR, \( K_{cz} \), FOR HORIZONTAL CURVE

Figure 49-2B
$CZ_t =$ clear-zone width on tangent section
$CZ_c =$ clear-zone width on horizontal curve
$L =$ transition length (ft) $= 3.1V$
$V =$ design speed (mph)

CLEAR-ZONE TRANSITION FOR CURVE ADJUSTMENT,  
RADIUS $\leq 3000$ ft

Figure 49-2C
CLEAR-ZONE TRANSITION FOR TANGENT SECTION OR CURVE WITH RADIUS > 3000 ft

Figure 49-2D
Find the average slope to Point A for opposing traffic:

\[
\frac{12 \times (-0.4) + 8 \times (-0.10) + 15 \times (-0.25)}{35} = \frac{-4.8 + (-0.8) + (-3.75)}{35} = 0.144 \text{ or } 7:1
\]

Find the average slope to Point A for adjacent traffic:

\[
\frac{8 \times (-0.10) + 15 \times (-0.25)}{23} = \frac{(-0.8) + (-3.75)}{23} = 0.20 \text{ or } 5:1
\]

Slope Average is 5:1

SLOPE-AVERAGING EXAMPLE

Figure 49-2E
CLEAR-ZONE APPLICATION FOR NON-RECOVERABLE FILL SLOPE

Figure 49-2F

* - IF THIS SLOPE IS STEEPER THAN 4:1, THE CLEAR-ZONE DISTANCE SHOULD BE BASED ON THE SLOPE OF THE SHOULDER

L = LENGTH OF RUNOUT AREA REQ'D AT TOE OF NON-RECOVERABLE SLOPE

L or 12' (WHICHEVER IS GREATER)

CLEAR RUNOUT AREA

TRAVERSABLE DITCH CROSS SECTIONS IS ACCEPTABLE IN THE RUNOUT AREA

* - IF THIS SLOPE IS STEEPER THAN 4:1, THE CLEAR-ZONE DISTANCE SHOULD BE BASED ON THE SLOPE OF THE SHOULDER
TYPICAL CUT SECTION

TYPICAL FILL SECTION

CLEAR-ZONE APPLICATION FOR SIDE SLOPE ON NEW FACILITY

Figure 49-2G
CLEAR-ZONE APPLICATION FOR CUT SLOPE
(2:1 Backslope)

Figure 49-2H
CLEAR-ZONE APPLICATION
FOR AUXILIARY LANE OR RAMP

Figure 49-2 I
CLEAR-ZONE / SLOPE AVERAGE,
EXAMPLE 49-2.2

Figure 49-2J
CLEAR-ZONE SLOPE AVERAGE
EXAMPLE 49-2.3

Figure 49-2K
APPURtenance-Free Zone

Figure 49-2L
<table>
<thead>
<tr>
<th>Facility</th>
<th>Design Speed, <em>V</em> (mph)</th>
<th>Design-Year AADT</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway or Other Divided Highway</td>
<td>All</td>
<td>All</td>
<td>10:1</td>
</tr>
<tr>
<td>Other Roadway</td>
<td>≤ 40</td>
<td>&lt; 12,000</td>
<td>4:1</td>
</tr>
<tr>
<td></td>
<td>≤ 40</td>
<td>≥ 12,000</td>
<td>6:1</td>
</tr>
<tr>
<td></td>
<td>45 or 50</td>
<td>All</td>
<td>6:1</td>
</tr>
<tr>
<td></td>
<td>≥ 55</td>
<td>&lt; 6,000</td>
<td>6:1</td>
</tr>
<tr>
<td></td>
<td>≥ 55</td>
<td>≥ 6,000</td>
<td>10:1</td>
</tr>
</tbody>
</table>

**Notes:**

1. This figure applies to a ditch check, median crossover, drive, or public-road approach.

2. A culvert within the clear zone under one of these embankments should have grated inlets and outlets, which are placed on a slope not steeper than shown above.

**TRANSVERSE SLOPES**

Figure 49-3A
BARRIER WARRANT FOR EMBANKMENT, 2-LANE, 2-WAY ROADWAY, 35 or 40 mph

Figure 49-3B (35,40)
BARRIER WARRANT FOR EMBANKMENT, 2-LANE, 2-WAY ROADWAY, 45 mph

Figure 49-3B (45)
BARRIER WARRANT FOR EMBANKMENT, 2-LANE, 2-WAY ROADWAY, 50 mph

Figure 49-3B (50)
BARRIER WARRANT FOR EMBANKMENT, 2-LANE, 2-WAY ROADWAY, 55 mph

Figure 49-3B (55)
BARRIER WARRANT FOR EMBANKMENT,
2-LANE, 2-WAY ROADWAY, 60 mph

Figure 49-3B (60)
BARRIER WARRANT FOR EMBANKMENT
2-LANE, 2-WAY ROADWAY, 70 mph

Figure 49-3B (70)
BARRIER WARRANT FOR EMBANKMENT: ROADWAY OF 4 OR MORE LANES; DIVIDED OR UNDIVIDED

Figure 49-3C
PREFERRED NARROW-WIDTH DITCH CROSS SECTION

V-Ditch with W = 0, Rounded Ditch with W < 8 ft, or Trapezoidal Ditch with W < 4 ft

Figure 49-3D
<table>
<thead>
<tr>
<th>Span</th>
<th>Rise</th>
<th>Option</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 10 ft</td>
<td>All</td>
<td>A</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>&lt; 66 in.</td>
<td>A</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>≥ 66 in.</td>
<td>B</td>
</tr>
</tbody>
</table>

A Establish a clear zone for a distance $L_R$ in advance of the structure. If this option is not cost-effective, guardrail should be placed.

B Guardrail should be placed.

CLEAR ZONE / GUARDRAIL AT CULVERT

Figure 49-3D(1)
PREFERRED MEDIUM-WIDTH DITCH CROSS SECTION
Rounded Ditch with $8 \text{ ft} \leq W \leq 12 \text{ ft}$ or Trapezoidal Ditch with $4 \text{ ft} \leq W \leq 8 \text{ ft}$

Figure 49-3E
PREFERRED WIDE-WIDTH DITCH CROSS SECTION
Rounded Ditch with W > 12 ft or Trapezoidal Ditch with W > 8ft

Figure 49-3F
LARGE CULVERT END WITHIN CLEAR-ZONE

Figure 49-3G
PIPE SIZE REQUIRED

CULVERT END TREATMENT, MEDIAN SECTION

Figure 49-3H
<table>
<thead>
<tr>
<th>Span</th>
<th>Rise</th>
<th>Option</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 10 ft</td>
<td>All</td>
<td>A</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>&lt; 66 in.</td>
<td>A</td>
</tr>
<tr>
<td>&gt; 10 ft</td>
<td>≥ 66 in.</td>
<td>B</td>
</tr>
</tbody>
</table>

A Establish a clear zone for a distance $L_R$ in advance of the structure. If this option is not cost-effective, a barrier should be placed.

B A barrier should be placed.

CLEAR ZONE / BARRIER AT CULVERT

Figure 49-3 I
DIVIDED HIGHWAY

UNDIVIDED HIGHWAY

A  GRATED BOX END SECTION TYPE I I
B  GRATED BOX END SECTION TYPE I I  OR  SAFETY METAL CULVERT END SECTION
C  STANDARD METAL CULVERT END SECTION

CULVERT END TREATMENT, LONGITUDINAL STRUCTURE

Figure 49-3J
(A) TANGENT SECTION
(Example 1)

(B) SUPERELEVATED SECTION
(Example 2)

BRIDGE PIER AND SPILLSLOPE CLEARANCE,
NEW CONSTRUCTION

Figure 49-3K
TREATMENT AT EXISTING BRIDGE CONE,
SLOPEWALL ≥ 30 FT FROM TRAVEL LANE

Figure 49-3L
TREATMENT AT EXISTING BRIDGE CONE
WITH SHOULDER PIER

Figure 49-3N
** A SPECIAL DETAIL FOR
ATTACHING THE RS-1A-A
SIGN IS REQUIRED IF SWS
EXCEEDS 42 in.

FIELD DRILL 4 HOLES
(INSIDE FLANGES) AND FASTEN
WITH 0.15 in.-0.30 in. X 1 in. BOLTS

RS-1A-A
(42 in. X 30 in.)

W

BACK OF EXIT
GORE SIGN

(USE TWO WBG
BREAKAWAY POSTS)

** GORE-AREA TREATMENT
Figure 49-3 O
BREAKAWAY SUPPORT STUB CLEARANCE DIAGRAM

Figure 49-3P
EDGE OF SHOULDER

2 ft MINIMUM
3 ft DESIRABLE
(OR 12 ft FROM
EDGE OF TRAVEL LANE,
WHICHEVER IS GREATER)

3 ft

20:1

EXISTING SLOPE

<table>
<thead>
<tr>
<th>EXISTING SLOPE</th>
<th>A (MAXIMUM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4:1</td>
<td>3:1</td>
</tr>
<tr>
<td>3:1</td>
<td>2:1</td>
</tr>
<tr>
<td>2:1</td>
<td>1.5:1</td>
</tr>
</tbody>
</table>

NOTE: LONGITUDINAL APPROACH AND EXIT SLOPES
SHOULD BE NO STEEPER THAN 20:1

LIGHT-STANDARD TREATMENT,
FILL SLOPE 4:1 OR STEEPER

Figure 49-3Q
SHOULDER WEDGES
Figure 49-3R
<table>
<thead>
<tr>
<th>TEST LEVEL</th>
<th>TYPE OF RAIL</th>
<th>CRASH TEST</th>
<th>MAXIMUM DYNAMIC DEFLECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>TL-3</td>
<td>Guardrail, 6’-3” Post Spacing</td>
<td>2</td>
<td>4.30 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, 3’ 1 ½” Post Spacing</td>
<td>2</td>
<td>3.30 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, 1’-6 ¾” Post Spacing</td>
<td>2</td>
<td>2.80 ft</td>
</tr>
<tr>
<td>TL-4</td>
<td>Guardrail, Thrie-Beam, 6’-3” Post Spacing</td>
<td>2</td>
<td>3.75 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Thrie-Beam, 3’ 1 ½” Post Spacing</td>
<td>1</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Thrie-Beam, 1’-6 ¾” Post Spacing</td>
<td>1</td>
<td>2.50 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Type B, 12’-6” Post Spacing</td>
<td>1</td>
<td>7.55 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Type B, 6’-3” Post Spacing</td>
<td>1</td>
<td>4.30 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Type B, 3’-1 ½” Post Spacing</td>
<td>1</td>
<td>3.30 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Type B, 1’-6 ¾” Post Spacing</td>
<td>1</td>
<td>2.80 ft</td>
</tr>
<tr>
<td>TL-3</td>
<td>Guardrail, Type B, 1’-6 ¾” Post Spacing</td>
<td>1</td>
<td>2.80 ft</td>
</tr>
<tr>
<td>³</td>
<td>Concrete Barrier</td>
<td>1</td>
<td>0.00 ft</td>
</tr>
</tbody>
</table>

**Notes:**

1. The crash test is designated as follows:
   1. Based on 4470-lb sedan, 60 mph, 25-deg impact angle
   2. Based on 4400-lb pickup, 62 mph, 25-deg impact angle

2. Maximum dynamic deflection width is measured from the front face of the barrier in its correct location to the front face of the barrier once it is deflected.

3. Concrete barrier of 2’-9” height is TL-4. Concrete barrier of 3’-9” height is TL-5.

**BARRIER DEFLECTIONS**

Figure 49-4A
TAPER RATE (SEE TABLE BELOW)

<table>
<thead>
<tr>
<th>DESIGN SPEED (mph)</th>
<th>TAPER RATE</th>
</tr>
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<tbody>
<tr>
<td>70</td>
<td>70:1</td>
</tr>
<tr>
<td>60</td>
<td>65:1</td>
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<td>55</td>
<td>60:1</td>
</tr>
<tr>
<td>50</td>
<td>50:1</td>
</tr>
<tr>
<td>45</td>
<td>45:1</td>
</tr>
<tr>
<td>40</td>
<td>40:1</td>
</tr>
</tbody>
</table>

$L_s =$ SHY LINE OFFSET (m), SEE FIGURE 49-4E

BARRIER PLACEMENT AT CURB

Figure 49-4B
BARRIER LENGTH OF NEED IN ADVANCE OF HAZARD

Figure 49-4D
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Runout Length, $L_R$ (ft)</th>
<th>Shy-Line Offset, $L_s$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T \leq 1000$</td>
<td>$1000 &lt; T \leq 5000$</td>
</tr>
<tr>
<td>30</td>
<td>70</td>
<td>80</td>
</tr>
<tr>
<td>40</td>
<td>100</td>
<td>110</td>
</tr>
<tr>
<td>45</td>
<td>125</td>
<td>135</td>
</tr>
<tr>
<td>50</td>
<td>150</td>
<td>160</td>
</tr>
<tr>
<td>55</td>
<td>175</td>
<td>185</td>
</tr>
<tr>
<td>60</td>
<td>200</td>
<td>210</td>
</tr>
<tr>
<td>70</td>
<td>250</td>
<td>290</td>
</tr>
</tbody>
</table>

*Note: This figure is in accordance with the suggested values from the AASHTO Roadside Design Guide 4th Edition 2011*

**DESIGN ELEMENTS FOR BARRIER LENGTH OF NEED**

Figure 49-4E [Rev. April 2013]
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>With GRET Type OS, MS, or II (ft)</th>
<th>With GRET Type I (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 50</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>≤ 45</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

MINIMUM GUARDRAIL LENGTH REQUIRED IN ADVANCE OF HAZARD

Figure 49-4E(1)
NOTES:

1. For a two-lane two-way roadway, this configuration should be used at all four corners.

2. For a one-way roadway, this configuration should be used only on the upstream-approach's outside shoulder. Guardrail is required on the downstream side only if there is a hazard to be shielded.

3. The \( L_{ET} \) portion of a guardrail end treatment type OS should be considered as part of \( L_N \) as described in Section 49-8.01(04) Item 2.

GUARDRAIL CONFIGURATION FOR OUTSIDE-SHOULDER APPROACH TO BRIDGE

Figure 49-4E (2)
GUARDRAIL CONFIGURATION AND LENGTH OF NEED FOR MEDIAN-SHOULDER APPROACH TO BRIDGE

Figure 49-4E (3)

NOTES:

1. For a one-way roadway, this configuration should be used only on the upstream-approach’s median shoulder. Guardrail is required on the downstream side only if there is a hazard to be shielded.

2. In the fractional values in the table, the numerator represents the clear-zone width, (ft). The denominator represents \( L_N \) (ft).

3. The \( L_{ET} \) portion of a guardrail end treatment type MS should be considered as part of \( L_N \) as described in Section 49-8.01(04) Item 2.
GUARDRAIL CONFIGURATION FOR BRIDGE SUPPORT INSIDE CLEAR ZONE, TWO-WAY ROADWAY, SINGLE OVERHEAD STRUCTURE

Figure 49-4E (4)
GUARDRAIL CONFIGURATION FOR BRIDGE SUPPORT INSIDE CLEAR ZONE, TWO-WAY ROADWAY, TWIN OVERHEAD STRUCTURES

Figure 49-4E (5)
GUARDRAIL CONFIGURATION FOR BRIDGE SUPPORT INSIDE CLEAR ZONE, ONE-WAY ROADWAY, SINGLE OVERHEAD STRUCTURE, OUTSIDE SHOULDER

Figure 49-4E (6)
GUARDRAIL CONFIGURATION FOR BRIDGE SUPPORT INSIDE CLEAR ZONE, ONE-WAY ROADWAY, TWIN OVERHEAD STRUCTURES, OUTSIDE SHOULDER

Figure 49-4E (7)
(a): >16 ft Edge of Travelway to Front Face of Bridge Support

(b): ≤16 ft Edge of Travelway to Front Face of Bridge Support

GUARDRAIL CONFIGURATION FOR BRIDGE SUPPORT INSIDE CLEAR ZONE, ONE-WAY ROADWAY, SINGLE OVERHEAD STRUCTURE, MEDIAN SHOULDER

Figure 49-4E (8)
Guardrail Configuration for Bridge Support Inside Clear Zone, One-Way Roadway, Twin Overhead Structures, Median Shoulder

(a): >16 ft Edge of Travelway to Front Face of Bridge Support

(b): ≤16 ft Edge of Travelway to Front Face of Bridge Support

Figure 49-4E (9)
<table>
<thead>
<tr>
<th>Travel Configuration</th>
<th>Overpass Type and Location</th>
<th>Guardrail Pay Length, Support &gt; 16 ft from Edge of Travelway (ft)</th>
<th>Guardrail Pay Length, Support ≤ 16 ft from Edge of Travelway (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-Way Roadway</td>
<td>One Structure</td>
<td>$2(L_N - L_{ET}) + L_P$</td>
<td>$2(L_N - L_{ET} + L_G)$</td>
</tr>
<tr>
<td></td>
<td>Twin Structures</td>
<td>$2(L_N - L_{ET} + L_P) + L_G$</td>
<td>$2(L_N - L_{ET} + L_G)$</td>
</tr>
<tr>
<td>One-Way Roadway</td>
<td>Outside Shoulder, One Structure</td>
<td>$L_N - L_{ET} + L_P + 25$</td>
<td>$L_N - L_{ET} + L_G$</td>
</tr>
<tr>
<td></td>
<td>Outside Shoulder, Twin Structures</td>
<td>$L_N - L_{ET} + 2L_P + L_G + 25$</td>
<td>$L_N - L_{ET} + L_G$</td>
</tr>
<tr>
<td></td>
<td>Median-Side Shoulder</td>
<td>(1)</td>
<td>$L_N - L_{ET} + L_G$</td>
</tr>
</tbody>
</table>

* (1) No guardrail is required. An impact attenuator is required where shown on Figure 49-4E(8) or 49-4E(9).

GUARDRAIL PAY LENGTH FOR APPROACH TO BRIDGE SUPPORT INSIDE CLEAR ZONE

Figure 49-4E(10)
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Inside Shy Line</th>
<th>Outside Shy Line</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>All Barriers</td>
<td>Concrete Barrier</td>
</tr>
<tr>
<td>30</td>
<td>13:1</td>
<td>8:1</td>
</tr>
<tr>
<td>35</td>
<td>16:1</td>
<td>10:1</td>
</tr>
<tr>
<td>40</td>
<td>17:1</td>
<td>11:1</td>
</tr>
<tr>
<td>45</td>
<td>18:1</td>
<td>12:1</td>
</tr>
<tr>
<td>50</td>
<td>21:1</td>
<td>14:1</td>
</tr>
<tr>
<td>55</td>
<td>24:1</td>
<td>16:1</td>
</tr>
<tr>
<td>60</td>
<td>26:1</td>
<td>18:1</td>
</tr>
<tr>
<td>65</td>
<td>30:1</td>
<td>20:1</td>
</tr>
</tbody>
</table>

**BARRIER FLARE RATES**

*Figure 49-4F*
Where:

- $L_1 = 25$ = Minimum length of tangent section of barrier upstream from hazard
- $L_2$ = Distance from ETL to tangent section of barrier
- $L_H$ = Distance from ETL to lateral extent of hazard
- $L_C$ = Clear zone width (See Figure 49-2A)
- $L_R$ = Runout length (See Figure 49-4E)
- ETL = Edge of travel lane

*B* – WHICHEVER IS LESS
HAZARD

E.T.L.

L_H or C_L

L_2

END OF BARRIER NEED

TANGENTIAL RUNOUT PATH

G.R.E.T.

L_R

BARRIER LAYOUT,
FIXED OBJECT ON HORIZONTAL CURVE

Figure 49-4H

* -WHICHEVER IS LESS

** -WHICHEVER REQUIRES LESS BARRIER LENGTH
GUARDRAIL LENGTH BEYOND HAZARD, 2-LANE ROADWAY

Figure 49-4 I
AREA OF CONCERN (HAZARD)

END OF BARRIER NEED

ACCEPTABLE ANCHORAGE WHICH IS 25 ft OF STANDARD GUARDRAIL WITH 6.25 ft POST SPACING

25°

E.T.L.

TRAFFIC

GUARDRAIL LENGTH BEYOND HAZARD, DIVIDED HIGHWAY

Figure 49-4J
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Pier End Inside Clear Zone</th>
<th>Pier End Outside Clear Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 50</td>
<td>Calculated Length of Need or 100 ft, whichever is greater</td>
<td>100 ft</td>
</tr>
<tr>
<td>≤ 45</td>
<td>Calculated Length of Need or 50 ft, whichever is greater</td>
<td>50 ft</td>
</tr>
</tbody>
</table>

LENGTH-OF-NEED REQUIREMENT FOR PIER PROTECTION

Figure 49-4K
BARRIER LENGTH OF NEED,
STRUCTURE-APPROACH EXAMPLE 49-4.1

Figure 49-4L
BARRIER LENGTH OF NEED,
FILL-SLOPE EXAMPLE 49-4.2

Figure 49-4M
<table>
<thead>
<tr>
<th>Test Level</th>
<th>Test Vehicle</th>
<th>Impact Speed (mph)</th>
<th>Impact Angle (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TL-2</td>
<td>4400-lb Pickup Truck</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>TL-3</td>
<td>4400-lb Pickup Truck</td>
<td>60</td>
<td>25</td>
</tr>
<tr>
<td>TL-4</td>
<td>18,000-lb Single-Unit Truck</td>
<td>50</td>
<td>15</td>
</tr>
<tr>
<td>TL-5</td>
<td>80,000-lb Tractor and Van Trailer</td>
<td>50</td>
<td>15</td>
</tr>
</tbody>
</table>

**NCHRP 350 TEST LEVEL CRASH-TEST CRITERIA**

Figure 49-5A
MEDIAN-BARRIER WARRANTS

Figure 49-6A
Grade Traffic-Adjustment Factor, $K_g$ and Curvature Traffic-Adjustment Factor, $K_c$

Figure 49-8B
DECK HEIGHT ABOVE UNDER-STRUCTURE SURFACE (ft)

TRAFFIC-ADJUSTMENT FACTOR $K_s$,
Deck Height and Under-Structure Shoulder Height Conditions

Figure 49-6C
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction Year Average Annual Daily Traffic, T, (1000’s) for Traffic Barrier Test Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Highway Type</td>
</tr>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
</tr>
<tr>
<td></td>
<td>Undivided With 4 Lanes or Fewer</td>
</tr>
<tr>
<td></td>
<td>One-Way</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
</tr>
<tr>
<td>% Trk</td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>T-2</td>
</tr>
<tr>
<td>0≤ %&lt;5</td>
<td></td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt;90.4</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt;148.3</td>
</tr>
<tr>
<td>&gt;12</td>
<td>&lt;316.0</td>
</tr>
<tr>
<td>5≤ %&lt;10</td>
<td></td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt;36.5</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt;55.9</td>
</tr>
<tr>
<td>&gt;12</td>
<td>&lt;100.7</td>
</tr>
<tr>
<td>10≤ %&lt;15</td>
<td></td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt;22.8</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt;34.4</td>
</tr>
<tr>
<td>&gt;12</td>
<td>&lt;59.9</td>
</tr>
<tr>
<td>15≤ %&lt;20</td>
<td></td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt;16.6</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt;24.9</td>
</tr>
<tr>
<td>&gt;12</td>
<td>&lt;42.6</td>
</tr>
<tr>
<td>20≤ %&lt;25</td>
<td></td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt;8.7</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt;19.5</td>
</tr>
<tr>
<td>&gt;12</td>
<td>&lt;33.1</td>
</tr>
</tbody>
</table>

**MEDIAN BARRIER AND BRIDGE RAILING TEST LEVEL SELECTION**

DESIGN SPEED 30 mph

Figure 49-6D(30)
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction Year Average Annual Daily Traffic, T, (1000’s) for Traffic Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Trk.</td>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>25 ≤ % &lt; 30</td>
<td>≤ 3</td>
<td>&lt; 7.2</td>
</tr>
<tr>
<td>3 ≤ L2 ≤ 7</td>
<td>&lt; 10.8</td>
<td>10.8 ≤ T &lt; 63.8</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 16.0</td>
<td>16.0 ≤ T &lt; 99.1</td>
</tr>
<tr>
<td>30 ≤ % &lt; 35</td>
<td>≤ 3</td>
<td>&lt; 6.1</td>
</tr>
<tr>
<td>3 ≤ L2 ≤ 7</td>
<td>&lt; 9.2</td>
<td>9.2 ≤ T &lt; 53.7</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 13.6</td>
<td>13.6 ≤ T &lt; 83.4</td>
</tr>
<tr>
<td>35 ≤ % ≤ 40</td>
<td>≤ 3</td>
<td>&lt; 5.3</td>
</tr>
<tr>
<td>3 ≤ L2 ≤ 7</td>
<td>&lt; 8.0</td>
<td>8.0 ≤ T &lt; 46.4</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 11.8</td>
<td>11.8 ≤ T &lt; 72.0</td>
</tr>
<tr>
<td></td>
<td>≥ 14 ≤ T &lt; 148.9</td>
<td>≥ 148.9</td>
</tr>
</tbody>
</table>

**MEDIAN BARRIER AND BRIDGE RAILING TEST LEVEL SELECTION**

**DESIGN SPEED 30 mph** (Continued)

*Figure 49-6D(30)*
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, $T_c$ (1000s) for Traffic-BARRIER Test Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Travel Lane to Front Face Barrier, $L_2$ (ft)</td>
<td><strong>Highway Type</strong></td>
</tr>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
</tr>
<tr>
<td><strong>0≤ % Trk &lt; 5</strong></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>14.0</td>
</tr>
<tr>
<td>3 ≤ $L_2$ ≤ 7</td>
<td>18.0</td>
</tr>
<tr>
<td>7 ≤ $L_2$ ≤ 12</td>
<td>24.4</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>39.5</td>
</tr>
<tr>
<td><strong>5≤ % Trk &lt; 10</strong></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>9.8</td>
</tr>
<tr>
<td>3 ≤ $L_2$ ≤ 7</td>
<td>12.7</td>
</tr>
<tr>
<td>7 ≤ $L_2$ ≤ 12</td>
<td>16.9</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>25.8</td>
</tr>
<tr>
<td><strong>10≤ % Trk &lt; 15</strong></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>7.5</td>
</tr>
<tr>
<td>3 ≤ $L_2$ ≤ 7</td>
<td>9.8</td>
</tr>
<tr>
<td>7 ≤ $L_2$ ≤ 12</td>
<td>12.9</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>19.1</td>
</tr>
<tr>
<td><strong>15≤ % Trk &lt; 20</strong></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>6.1</td>
</tr>
<tr>
<td>3 ≤ $L_2$ ≤ 7</td>
<td>8.0</td>
</tr>
<tr>
<td>7 ≤ $L_2$ ≤ 12</td>
<td>10.4</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>15.2</td>
</tr>
<tr>
<td><strong>20≤ % Trk &lt; 25</strong></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>5.1</td>
</tr>
<tr>
<td>3 ≤ $L_2$ ≤ 7</td>
<td>6.7</td>
</tr>
<tr>
<td>7 ≤ $L_2$ ≤ 12</td>
<td>8.8</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>12.6</td>
</tr>
</tbody>
</table>

**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 40 mph**

Figure 49-6D(40)
### MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 40 mph (Continued)

**Figure 49-6D(40)**

<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, ( T ), (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Trk.</td>
<td>Divided, or Undivided With 5 or More Lanes Test Level</td>
<td>Undivided With 4 Lanes or Fewer Test Level</td>
</tr>
<tr>
<td>25 ≤ % &lt;30</td>
<td>3 ≤ 3</td>
<td>4.4 ≤ ( T &lt; 20.6 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>3 ≤ L2 ≤ 7</td>
<td>5.8 ≤ ( T &lt; 22.9 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>7 ≤ L2 ≤ 12</td>
<td>7.5 ≤ ( T &lt; 34.6 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>&gt;12</td>
<td>10.8 ≤ ( T &lt; 46.1 )</td>
</tr>
<tr>
<td>30 ≤ % &lt;35</td>
<td>3 ≤ 3</td>
<td>3.9 ≤ ( T &lt; 17.4 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>3 ≤ L2 ≤ 7</td>
<td>5.1 ≤ ( T &lt; 19.3 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>3 ≤ L2 ≤ 7</td>
<td>6.6 ≤ ( T &lt; 29.2 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>&gt;12</td>
<td>9.4 ≤ ( T &lt; 38.8 )</td>
</tr>
<tr>
<td>35 ≤ % ≤40</td>
<td>3 ≤ 3</td>
<td>3.5 ≤ ( T &lt; 15.0 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>3 ≤ L2 ≤ 7</td>
<td>4.6 ≤ ( T &lt; 16.7 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>7 ≤ L2 ≤ 12</td>
<td>5.9 ≤ ( T &lt; 25.3 )</td>
</tr>
<tr>
<td>% Trk.</td>
<td>&gt;12</td>
<td>8.4 ≤ ( T &lt; 33.5 )</td>
</tr>
<tr>
<td>Site Characteristics</td>
<td>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-BARRIER Test Levels</td>
<td>Highway Type</td>
</tr>
<tr>
<td>----------------------</td>
<td>--------------------------------------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>% Trk ≤ 3</td>
<td>Divided, or Undivided With 5 or More Lanes</td>
<td></td>
</tr>
<tr>
<td>Barrie, L2 (ft) to Front Face Travel Lane</td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>0 ≤ % L2 &lt; 5</td>
<td>&lt; 9.7</td>
<td>9.7 ≤ T &lt; 221.4</td>
</tr>
<tr>
<td>5 ≤ % L2 &lt; 10</td>
<td>&lt; 12.1</td>
<td>12.1 ≤ T &lt; 261.8</td>
</tr>
<tr>
<td>10 ≤ % L2 &lt; 15</td>
<td>&lt; 16.4</td>
<td>16.4 ≤ T &lt; 349.6</td>
</tr>
<tr>
<td>15 ≤ % L2 &lt; 20</td>
<td>&lt; 25.3</td>
<td>≥ 25.3</td>
</tr>
<tr>
<td>20 ≤ % L2 &lt; 25</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>One-Way</td>
<td>Divided, or Undivided With 4 Lanes or Fewer</td>
<td></td>
</tr>
<tr>
<td>Barrie, L2 (ft) to Front Face Travel Lane</td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>0 ≤ % L2 &lt; 5</td>
<td>&lt; 9.7</td>
<td>9.7 ≤ T &lt; 221.4</td>
</tr>
<tr>
<td>5 ≤ % L2 &lt; 10</td>
<td>&lt; 12.1</td>
<td>12.1 ≤ T &lt; 261.8</td>
</tr>
<tr>
<td>10 ≤ % L2 &lt; 15</td>
<td>&lt; 16.4</td>
<td>16.4 ≤ T &lt; 349.6</td>
</tr>
<tr>
<td>15 ≤ % L2 &lt; 20</td>
<td>&lt; 25.3</td>
<td>≥ 25.3</td>
</tr>
<tr>
<td>20 ≤ % L2 &lt; 25</td>
<td>0.4</td>
<td>0.4</td>
</tr>
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</table>

**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 45 mph**

*Figure 49-6D(45)*
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, $T$, (1000s) for Traffic-Barrier Test Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Trk.</td>
<td>Highway Type</td>
</tr>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes Test Level</td>
</tr>
<tr>
<td>25≤ % Trk. &lt;30</td>
<td></td>
</tr>
<tr>
<td>3 ≤ L₂ ≤ 7</td>
<td>&lt; 3.7</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 4.6</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>30≤ % Trk. &lt;35</td>
<td></td>
</tr>
<tr>
<td>3 ≤ L₂ ≤ 7</td>
<td>&lt; 3.3</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 4.1</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>35≤ % Trk. ≤40</td>
<td></td>
</tr>
<tr>
<td>3 ≤ L₂ ≤ 7</td>
<td>&lt; 3.0</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 3.8</td>
</tr>
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**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 45 mph** (Continued)

*Figure 49-6D(45)*
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Trk.</td>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test Level</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 5.5</td>
<td>5.5 ≤ T &lt; 162.2</td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt; 6.3</td>
<td>6.3 ≤ T &lt; 188.6</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt; 8.4</td>
<td>8.4 ≤ T &lt; 247.3</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 11.2</td>
<td>11.2 ≤ T &lt; 314.7</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 4.7</td>
<td>4.7 ≤ T &lt; 50.0</td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt; 5.4</td>
<td>5.4 ≤ T &lt; 61.4</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt; 7.2</td>
<td>7.2 ≤ T &lt; 70.6</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 9.6</td>
<td>9.6 ≤ T &lt; 88.5</td>
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<tr>
<td>≤ 3</td>
<td>&lt; 4.1</td>
<td>4.1 ≤ T &lt; 29.6</td>
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<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt; 4.8</td>
<td>4.8 ≤ T &lt; 36.7</td>
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<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt; 6.3</td>
<td>6.3 ≤ T &lt; 41.2</td>
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<tr>
<td>&gt; 12</td>
<td>&lt; 8.4</td>
<td>8.4 ≤ T &lt; 51.5</td>
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<tr>
<td>≤ 3</td>
<td>&lt; 3.7</td>
<td>3.7 ≤ T &lt; 21.0</td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt; 4.3</td>
<td>4.3 ≤ T &lt; 26.1</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt; 5.6</td>
<td>5.6 ≤ T &lt; 29.1</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 7.5</td>
<td>7.5 ≤ T &lt; 36.3</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 3.3</td>
<td>3.3 ≤ T &lt; 16.3</td>
</tr>
<tr>
<td>3 &lt; L2 ≤ 7</td>
<td>&lt; 3.9</td>
<td>3.9 ≤ T &lt; 20.3</td>
</tr>
<tr>
<td>7 &lt; L2 ≤ 12</td>
<td>&lt; 5.0</td>
<td>5.0 ≤ T &lt; 22.5</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 6.7</td>
<td>6.7 ≤ T &lt; 28.1</td>
</tr>
</tbody>
</table>

MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 50 mph

Figure 49-6D(50)
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes Test Level</td>
<td>Undivided With 4 Lanes or Fewer Test Level</td>
</tr>
<tr>
<td>Edge of Travel Lane to Front Face Barrier, L₂ (ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 3.1</td>
<td>3.1 ≤ T &lt; 13.3</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 3.5</td>
<td>3.5 ≤ T &lt; 16.6</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 4.5</td>
<td>4.5 ≤ T &lt; 18.3</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 6.1</td>
<td>6.1 ≤ T &lt; 22.9</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 2.8</td>
<td>2.8 ≤ T &lt; 11.2</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 3.2</td>
<td>3.2 ≤ T &lt; 14.0</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 4.2</td>
<td>4.2 ≤ T &lt; 15.5</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 5.6</td>
<td>5.6 ≤ T &lt; 19.3</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 2.6</td>
<td>2.6 ≤ T &lt; 9.7</td>
</tr>
<tr>
<td>≤ 3</td>
<td>&lt; 3.0</td>
<td>3.0 ≤ T &lt; 12.2</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 3.8</td>
<td>3.8 ≤ T &lt; 13.4</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>&lt; 5.2</td>
<td>5.2 ≤ T &lt; 16.7</td>
</tr>
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**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 50 mph** (Continued)

**Figure 49-6D(50)**
<table>
<thead>
<tr>
<th>% Trk.</th>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>% Trk.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0&lt;</td>
<td>≤ 3</td>
<td>TL-2: 4.2 ≤ T &lt; 134.7 134.7 ≤ T &lt; 286.4 286.4 ≤ T &lt; 709.4 709.4 ≤ T &lt; 1346.8 1346.8 ≤ T &lt; 2004.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>TL-2: 4.2 ≤ T &lt; 157.4 157.4 ≤ T &lt; 314.8 314.8 ≤ T &lt; 629.6 629.6 ≤ T &lt; 1259.2 1259.2 ≤ T &lt; 2004.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>TL-2: 6.2 ≤ T &lt; 202.8 202.8 ≤ T &lt; 405.6 405.6 ≤ T &lt; 811.2 811.2 ≤ T &lt; 1622.4 1622.4 ≤ T &lt; 2552.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>TL-2: 8.1 ≤ T &lt; 259.2 259.2 ≤ T &lt; 518.4 518.4 ≤ T &lt; 1036.8 1036.8 ≤ T &lt; 1755.2 1755.2 ≤ T &lt; 2673.6</td>
<td></td>
</tr>
<tr>
<td>5&lt;</td>
<td>≤ 3</td>
<td>TL-2: 3.7 ≤ T &lt; 44.8 44.8 ≤ T &lt; 89.6 89.6 ≤ T &lt; 179.2 179.2 ≤ T &lt; 358.4 358.4 ≤ T &lt; 537.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>TL-2: 4.2 ≤ T &lt; 54.4 54.4 ≤ T &lt; 109.6 109.6 ≤ T &lt; 219.2 219.2 ≤ T &lt; 438.4 438.4 ≤ T &lt; 657.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>TL-2: 5.5 ≤ T &lt; 61.8 61.8 ≤ T &lt; 123.6 123.6 ≤ T &lt; 247.2 247.2 ≤ T &lt; 494.4 494.4 ≤ T &lt; 741.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>TL-2: 7.1 ≤ T &lt; 78.0 78.0 ≤ T &lt; 156.0 156.0 ≤ T &lt; 312.0 312.0 ≤ T &lt; 624.0 624.0 ≤ T &lt; 936.0</td>
<td></td>
</tr>
<tr>
<td>10&lt;</td>
<td>≤ 3</td>
<td>TL-2: 3.4 ≤ T &lt; 26.9 26.9 ≤ T &lt; 53.8 53.8 ≤ T &lt; 107.6 107.6 ≤ T &lt; 215.2 215.2 ≤ T &lt; 323.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>TL-2: 3.8 ≤ T &lt; 33.0 33.0 ≤ T &lt; 66.0 66.0 ≤ T &lt; 132.0 132.0 ≤ T &lt; 264.0 264.0 ≤ T &lt; 416.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>TL-2: 5.0 ≤ T &lt; 36.5 36.5 ≤ T &lt; 73.0 73.0 ≤ T &lt; 146.0 146.0 ≤ T &lt; 292.0 292.0 ≤ T &lt; 436.0</td>
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</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>TL-2: 6.4 ≤ T &lt; 46.0 46.0 ≤ T &lt; 92.0 92.0 ≤ T &lt; 184.0 184.0 ≤ T &lt; 368.0 368.0 ≤ T &lt; 552.0</td>
<td></td>
</tr>
<tr>
<td>15&lt;</td>
<td>≤ 3</td>
<td>TL-2: 3.1 ≤ T &lt; 19.2 19.2 ≤ T &lt; 38.4 38.4 ≤ T &lt; 76.8 76.8 ≤ T &lt; 153.6 153.6 ≤ T &lt; 230.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>TL-2: 3.5 ≤ T &lt; 23.6 23.6 ≤ T &lt; 47.2 47.2 ≤ T &lt; 94.4 94.4 ≤ T &lt; 188.8 188.8 ≤ T &lt; 283.2</td>
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<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>TL-2: 4.5 ≤ T &lt; 25.9 25.9 ≤ T &lt; 51.8 51.8 ≤ T &lt; 103.6 103.6 ≤ T &lt; 207.2 207.2 ≤ T &lt; 310.4</td>
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</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>TL-2: 5.8 ≤ T &lt; 32.6 32.6 ≤ T &lt; 65.2 65.2 ≤ T &lt; 130.4 130.4 ≤ T &lt; 260.8 260.8 ≤ T &lt; 421.6</td>
<td></td>
</tr>
<tr>
<td>20&lt;</td>
<td>≤ 3</td>
<td>TL-2: 2.8 ≤ T &lt; 15.0 15.0 ≤ T &lt; 30.0 30.0 ≤ T &lt; 60.0 60.0 ≤ T &lt; 120.0 120.0 ≤ T &lt; 180.0</td>
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</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>TL-2: 3.2 ≤ T &lt; 18.4 18.4 ≤ T &lt; 36.8 36.8 ≤ T &lt; 73.6 73.6 ≤ T &lt; 147.2 147.2 ≤ T &lt; 220.8</td>
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</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>TL-2: 4.1 ≤ T &lt; 20.1 20.1 ≤ T &lt; 40.2 40.2 ≤ T &lt; 80.4 80.4 ≤ T &lt; 160.8 160.8 ≤ T &lt; 241.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>TL-2: 5.3 ≤ T &lt; 25.3 25.3 ≤ T &lt; 50.6 50.6 ≤ T &lt; 101.2 101.2 ≤ T &lt; 202.4 202.4 ≤ T &lt; 303.6</td>
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</tr>
</tbody>
</table>

**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 55 mph**

Figure 49-6D(55)
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-BARRIER Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Trk. Edge of Travel Lane to Front Face Barrier, L₂ (ft)</td>
<td>Divided, or Undivided With 5 or More Lanes Test Level</td>
<td>Undivided With 4 Lanes or Fewer Test Level</td>
</tr>
<tr>
<td>25≤ % &lt;30</td>
<td>≤ 3</td>
<td>&lt; 2.7</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 3.0</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>&lt; 3.8</td>
</tr>
<tr>
<td>30≤ % &lt;35</td>
<td>≤ 3</td>
<td>&lt; 2.5</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 2.8</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>&lt; 3.6</td>
</tr>
<tr>
<td>35≤ % ≤40</td>
<td>≤ 3</td>
<td>&lt; 2.4</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L₂ ≤ 7</td>
<td>&lt; 2.6</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>&lt; 3.3</td>
</tr>
</tbody>
</table>

MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 55 mph (Continued)

Figure 49-6D(55)
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Highway Type</th>
<th>Divided, or Undivided With 5 or More Lanes</th>
<th>Undivided With 4 Lanes or Fewer</th>
<th>One-Way</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge of Travel Lane to Front Face Barrier, L2 (ft)</td>
<td>Test Level</td>
<td>Test Level</td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>0&lt;%&lt;5</td>
<td>≤ 3</td>
<td>n/a</td>
<td>3.0 ≤ T &lt; 107.3</td>
<td>1.9 ≤ T &lt; 70.3</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>n/a</td>
<td>3.3 ≤ T &lt; 126.3</td>
<td>2.1 ≤ T &lt; 82.8</td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>n/a</td>
<td>4.1 ≤ T &lt; 158.4</td>
<td>2.7 ≤ T &lt; 105.6</td>
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<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
<td>5.0 ≤ T &lt; 203.8</td>
<td>3.3 ≤ T &lt; 138.2</td>
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<tr>
<td>5&lt;%&lt;10</td>
<td>≤ 3</td>
<td>n/a</td>
<td>2.8 ≤ T &lt; 39.6</td>
<td>1.8 ≤ T &lt; 25.0</td>
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<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>n/a</td>
<td>3.1 ≤ T &lt; 47.5</td>
<td>2.0 ≤ T &lt; 29.3</td>
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<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>n/a</td>
<td>3.9 ≤ T &lt; 53.1</td>
<td>2.5 ≤ T &lt; 33.7</td>
</tr>
<tr>
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<td>&gt; 12</td>
<td>n/a</td>
<td>4.7 ≤ T &lt; 67.6</td>
<td>3.1 ≤ T &lt; 44.1</td>
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<tr>
<td>10&lt;%&lt;15</td>
<td>≤ 3</td>
<td>n/a</td>
<td>2.7 ≤ T &lt; 24.3</td>
<td>1.7 ≤ T &lt; 15.2</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>n/a</td>
<td>2.9 ≤ T &lt; 29.3</td>
<td>1.9 ≤ T &lt; 17.8</td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>n/a</td>
<td>3.7 ≤ T &lt; 31.9</td>
<td>2.4 ≤ T &lt; 20.0</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
<td>4.5 ≤ T &lt; 40.5</td>
<td>2.9 ≤ T &lt; 26.2</td>
</tr>
<tr>
<td>15&lt;%&lt;20</td>
<td>≤ 3</td>
<td>n/a</td>
<td>2.5 ≤ T &lt; 17.5</td>
<td>1.6 ≤ T &lt; 10.9</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>n/a</td>
<td>2.8 ≤ T &lt; 21.1</td>
<td>1.8 ≤ T &lt; 12.8</td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>n/a</td>
<td>3.5 ≤ T &lt; 22.8</td>
<td>2.2 ≤ T &lt; 14.3</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
<td>4.2 ≤ T &lt; 28.9</td>
<td>2.8 ≤ T &lt; 18.7</td>
</tr>
<tr>
<td>20&lt;%&lt;25</td>
<td>≤ 3</td>
<td>n/a</td>
<td>2.4 ≤ T &lt; 13.7</td>
<td>1.5 ≤ T &lt; 8.5</td>
</tr>
<tr>
<td></td>
<td>3 &lt; L2 ≤ 7</td>
<td>n/a</td>
<td>2.6 ≤ T &lt; 16.5</td>
<td>1.7 ≤ T &lt; 10.0</td>
</tr>
<tr>
<td></td>
<td>7 &lt; L2 ≤ 12</td>
<td>n/a</td>
<td>3.3 ≤ T &lt; 17.7</td>
<td>2.1 ≤ T &lt; 11.1</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
<td>4.0 ≤ T &lt; 22.5</td>
<td>2.6 ≤ T &lt; 14.5</td>
</tr>
</tbody>
</table>

MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 60 mph

Figure 49-6D(60)
<table>
<thead>
<tr>
<th>% Trk.</th>
<th>Test Level</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
<td>Undivided With 4 Lanes or Fewer</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>25≤</td>
<td>≤ 3</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>3 ≤ L2 ≤ 7</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
</tr>
<tr>
<td>30≤</td>
<td>≤ 3</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>3 ≤ L2 ≤ 7</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
</tr>
<tr>
<td>35≤</td>
<td>≤ 3</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>3 ≤ L2 ≤ 7</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>&gt; 12</td>
<td>n/a</td>
</tr>
<tr>
<td>40≤</td>
<td>≤ 3</td>
<td>n/a</td>
</tr>
</tbody>
</table>

**MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 60 mph** (Continued)

Figure 49-6D(60)
<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes</td>
<td>Undivided With 4 Lanes or Fewer</td>
</tr>
<tr>
<td></td>
<td>Test Level</td>
<td>Test Level</td>
</tr>
<tr>
<td>% Trk. ≤ 3</td>
<td>n/a</td>
<td>2.1 ≤ T &lt; 63.1</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>n/a</td>
<td>2.3 ≤ T &lt; 80.0</td>
</tr>
<tr>
<td>7 &lt; L₂ ≤ 12</td>
<td>n/a</td>
<td>2.7 ≤ T &lt; 96.4</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>n/a</td>
<td>3.1 ≤ T &lt; 127.6</td>
</tr>
<tr>
<td>% Trk. ≤ 3</td>
<td>n/a</td>
<td>2.0 ≤ T &lt; 32.0</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>n/a</td>
<td>2.3 ≤ T &lt; 38.5</td>
</tr>
<tr>
<td>7 &lt; L₂ ≤ 12</td>
<td>n/a</td>
<td>2.6 ≤ T &lt; 42.2</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>n/a</td>
<td>3.0 ≤ T &lt; 53.0</td>
</tr>
<tr>
<td>% Trk. ≤ 3</td>
<td>n/a</td>
<td>2.0 ≤ T &lt; 21.5</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>n/a</td>
<td>2.2 ≤ T &lt; 25.3</td>
</tr>
<tr>
<td>7 &lt; L₂ ≤ 12</td>
<td>n/a</td>
<td>2.6 ≤ T &lt; 27.0</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>n/a</td>
<td>3.0 ≤ T &lt; 33.5</td>
</tr>
<tr>
<td>% Trk. ≤ 3</td>
<td>n/a</td>
<td>1.9 ≤ T &lt; 16.2</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>n/a</td>
<td>2.1 ≤ T &lt; 18.9</td>
</tr>
<tr>
<td>7 &lt; L₂ ≤ 12</td>
<td>n/a</td>
<td>2.5 ≤ T &lt; 19.9</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>n/a</td>
<td>2.9 ≤ T &lt; 24.4</td>
</tr>
<tr>
<td>% Trk. ≤ 3</td>
<td>n/a</td>
<td>1.9 ≤ T &lt; 13.0</td>
</tr>
<tr>
<td>3 &lt; L₂ ≤ 7</td>
<td>n/a</td>
<td>2.0 ≤ T &lt; 15.1</td>
</tr>
<tr>
<td>7 &lt; L₂ ≤ 12</td>
<td>n/a</td>
<td>2.5 ≤ T &lt; 15.7</td>
</tr>
<tr>
<td>&gt; 12</td>
<td>n/a</td>
<td>2.8 ≤ T &lt; 19.2</td>
</tr>
</tbody>
</table>

MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION, DESIGN SPEED 65 mph or 70 mph

Figure 49-6D(65, 70)
## MEDIAN-BARRIER OR BRIDGE-RAILING TEST-LEVEL SELECTION,
### DESIGN SPEED 65 mph or 70 mph (Continued)

**Figure 49-6D(65, 70)**

<table>
<thead>
<tr>
<th>% Trk.</th>
<th>Edge of Travel Lane to Front Face Barrier, L₂ (ft)</th>
<th>Site Characteristics</th>
<th>Adjusted Construction-Year Average Annual Daily Traffic, T, (1000s) for Traffic-Barrier Test Levels</th>
<th>Highway Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Divided, or Undivided With 5 or More Lanes Test Level</td>
<td></td>
</tr>
<tr>
<td>25≤ %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt;30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td>30≤ %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt;35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td>35≤ %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>≤40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&lt; 12</td>
<td></td>
<td></td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
ELEVATION - NORTHBOUND LANES
TRUCK-HEIGHT CONCRETE MEDIAN-BARRIER
EXAMPLE 49-6.1

Figure 49-6E
CUTOFF ANGLE FOR GLARE SCREEN

Figure 49-6F
Wall height = 33 in. Min. above ground line

Ground Line

Bottom of footing 36 in. min. below ground line

48 in. Min.

Minimum Reinforcing Steel

<table>
<thead>
<tr>
<th>Type</th>
<th>Wall</th>
<th>Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Horizontal</td>
<td>#4 @ 12 in.</td>
<td>#4 @ 12 in. (At top of wall)</td>
</tr>
<tr>
<td>Vertical</td>
<td>#5 @ 12 in. (Both faces)</td>
<td>N.A.</td>
</tr>
</tbody>
</table>

COLLISION-WALL DETAIL

Figure 49-7A
GUARDRAIL END TREATMENT
TYPE OS OR MS
FOR CURVED GUARDRAIL RUN

Figure 49-8A
CLEAR ZONE DISTANCE OR
RIGHT-OF-WAY LINE
(WHICHEVER IS LESS)

LENGTH OF NEED

AREA OF CONCERN
(HAZARD)

END OF BARRIER
NEED

CRASHWORTHY
TERMINAL

E.T.L.

TRAFFIC

- FIXED OBJECT HAZARD
IN THIS AREA TO BE CHECKED
TO ASSURE THAT GUARDRAIL
LENGTH OF NEED IN ADVANCE
OF IT IS ADEQUATE.

- AREA WHICH MUST BE CLEAR
AND TRAVERSABLE FOR A
GIVEN RUN OF GUARDRAIL. THIS
INCLUDES A PATH THROUGH AND
BEHIND THE G.R.E.T.

CLEAR RECOVERY AREA BEHIND GUARDRAIL

Figure 49-8B
For an outside obstruction with no pavement located to the outside of the obstruction, D2 is defined to be greater than 50 ft.

MULTI-LANE DIVIDED FACILITY OFFSETS

IMPACT-ATTENUATOR OFFSETS

Figure 49-8C
<table>
<thead>
<tr>
<th>Attenuator Type</th>
<th>Test Level 3 (TL-3)</th>
<th>Test Level 2 (TL-2)</th>
<th>Test Level 1 (TL-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Offset Dimension $D1$ or $D2$ (if applicable)</td>
<td>Offset Dimension $D1$ or $D2$ (if applicable)</td>
<td>Offset Dimension $D1$ or $D2$ (if applicable)</td>
</tr>
</tbody>
</table>
| ED              | $* 25 \text{ ft} \leq D1 \leq 50 \text{ ft}$  
$25 \text{ ft} \leq D2 \leq 50 \text{ ft}$ | $* D1 = 25 \text{ ft}$  
$D2 = 25 \text{ ft}$ | n/a |
| R1              | $10 \text{ ft} < D1 \leq 50 \text{ ft}$  
$D2 > 50 \text{ ft}$ | $10 \text{ ft} < D1 \leq 25 \text{ ft}$  
$D2 > 25 \text{ ft}$ | n/a |
| R2              | $10 \text{ ft} < D1 < 25 \text{ ft}$  
$D2 \leq 50 \text{ ft}$ | $10 \text{ ft} < D1 < 25 \text{ ft}$  
$D2 \leq 25 \text{ ft}$ | n/a |
| CR              | $D1 \leq 10 \text{ ft}$  
$D2 \leq 50 \text{ ft}$ | $D1 \leq 10 \text{ ft}$  
$D2 \leq 25 \text{ ft}$ | n/a |
| LS              | n/a | n/a | $D1 \leq 18 \text{ ft}$ |
| None Required   | $D1 > 50 \text{ ft}$  
$D1 > 25 \text{ ft}$ | $D1 > 25 \text{ ft}$  
$D1 > 18 \text{ ft}$ | |

Notes:

$D1 = $ Offset dimension from edge of obstruction face to edge of travel lane in the direction of travel under consideration.

$D2 = $ Offset dimension from edge of obstruction face to edge of travel lane on the opposite side of the obstruction, if applicable.

$D1$ and $D2$ are based upon clear-zone requirements. See Section 49-8.04(02) and Figure 49-8C for additional information regarding $D1$ and $D2$.

* The required $D1 \geq 25 \text{ ft}$ is for installation of impact attenuator Type ED, gravel barrel array only.

IMPACT-ATTENUATOR TYPE DETERMINATION

Figure 49-8D
### Required impact attenuator system space, ft

**Footprint: Length (L) x Width (W)**

<table>
<thead>
<tr>
<th>(Type of attenuator)/Width designation</th>
<th>Test Level-3 (TL-3)</th>
<th>Test Level-2 (TL-2)</th>
<th>Test Level-1 (TL-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ED)/W1</td>
<td>48 x 20</td>
<td>30 x 16.5</td>
<td>N/A</td>
</tr>
<tr>
<td>(R1,R2,CR)/W1</td>
<td>36.5 x 4.5</td>
<td>18 x 4.5</td>
<td>N/A</td>
</tr>
<tr>
<td>(R1,R2,CR)/W2</td>
<td>36.5 x 8.5</td>
<td>18 x 8.5</td>
<td>N/A</td>
</tr>
<tr>
<td>(R1,R2,CR)/W3</td>
<td>36.5 x 10</td>
<td>18 x 10</td>
<td>N/A</td>
</tr>
<tr>
<td>(LS)/W1</td>
<td>N/A</td>
<td>N/A</td>
<td>10 x 4</td>
</tr>
</tbody>
</table>

**NOTES:**

1. The table shows approximate footprint of the required space, including pad, for the impact attenuator system.
2. Non-mountable curbs should not be used in front of impact attenuator.

**IMPACT-ATTENUATOR FOOTPRINT REQUIREMENTS**

**Figure 49-8E**
<table>
<thead>
<tr>
<th>Roadway and Overhead Structure Type</th>
<th>Pier Location</th>
<th>Pier to ETL Offset</th>
<th>Pier Protection Type</th>
<th>Collision Wall Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>TLTW, Single</td>
<td>Both Sides</td>
<td>≤ 16 ft</td>
<td>Guardrail w/ GP Trans.</td>
<td>Frame Bent Only</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;16 ft</td>
<td>Shoulder Guardrail</td>
<td>None</td>
</tr>
<tr>
<td>TLTW, Twin</td>
<td>Both Sides</td>
<td>≤ 16 ft</td>
<td>Guardrail w/ GP Trans.</td>
<td>Frame Bent &amp; Pier Gap</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;16 ft</td>
<td>Shoulder Guardrail</td>
<td>None</td>
</tr>
<tr>
<td>4LD, Single or Tandem</td>
<td>Outside</td>
<td>≤ 16 ft</td>
<td>Guardrail w/ GP Trans.</td>
<td>Frame Bent Only</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;16 ft</td>
<td>Shoulder Guardrail</td>
<td>None</td>
</tr>
<tr>
<td>4LD, Twin</td>
<td>Outside</td>
<td>≤ 16 ft</td>
<td>Guardrail w/ GP Trans.</td>
<td>Frame Bent &amp; Pier Gap</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;16 ft</td>
<td>Shoulder Guardrail</td>
<td>None</td>
</tr>
<tr>
<td>4LD, Single</td>
<td>Median Side</td>
<td>&lt; 25 ft</td>
<td>R2 Attenuator</td>
<td>Frame Bent Only</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 25 ft</td>
<td>ED Attenuator</td>
<td>Frame Bent Only</td>
</tr>
<tr>
<td>4LD, Twin</td>
<td>Median Side</td>
<td>&lt; 25 ft</td>
<td>R2 Attenuator</td>
<td>Frame Bent &amp; Pier Gap</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥ 25 ft</td>
<td>ED Attenuator</td>
<td>Frame Bent &amp; Pier Gap</td>
</tr>
<tr>
<td>4LD, Tandem</td>
<td>Median Side</td>
<td>≤ 16 ft</td>
<td>Guardrail w/ GP Trans.</td>
<td>Frame Bent Only</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt;16 ft</td>
<td>Shoulder Guardrail</td>
<td>None</td>
</tr>
</tbody>
</table>

Note: TLTW = Two-Lane, Two-Way roadway; 4LD = 4 or more Lanes, Divided; ETL = Edge of Travel Lane; Pier = Pier or frame bent; ED = Energy Dissipation; R2 = Redirective on 2 sides; GP Trans. = Guardrail to bridge Pier Transition. Shoulder guardrail should be used only where all clearance requirements are satisfied; otherwise, guardrail with GP transition should be used.

PIER-PROTECTION REQUIREMENTS

Figure 49-8F
<table>
<thead>
<tr>
<th>Design Speed, mph</th>
<th>With Guardrail End Treatment Type I</th>
<th>With Guardrail End Tmt. Type OS or MS</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 50</td>
<td>Length of Need or 100 ft, whichever is greater</td>
<td>Length of Need or 50 ft, whichever is greater</td>
</tr>
<tr>
<td>≤ 45</td>
<td>Length of Need or 50 ft, whichever is greater</td>
<td>Length of Need or 50 ft, whichever is greater</td>
</tr>
</tbody>
</table>

Note: This is the minimum bridge-approach guardrail length, including guardrail-transition length.

**BRIDGE-RAILING-END PROTECTION REQUIREMENTS**

*Figure 49-9A*
HAZARD POINT

G.R.E.T.

CURVED W-BEAM GUARDRAIL CONNECTOR SYSTEM

CONTROL LINE

TRAVERSABLE AREA

BRIDGE

E.T.L.

L_R

PUBLIC-ROAD APPROACH APPLICATION AT OR BEYOND THE CONTROL LINE

Figure 49-9B
HAZARD POINT

TYPE 5 ANCHOR

CURVED W-BEAM GUARDRAIL TERMINAL SYSTEM

TRAVERSABLE AREA

CONTROL LINE

E.T.L.

BRIDGE

L_R

DRIVE APPLICATION BEYOND THE CONTROL LINE

Figure 49-9D
NOTE:
IF $A \leq 100$ ft, OMIT GUARDRAIL AND TERMINAL SYSTEM IN ADVANCE OF DRIVE
IF $A > 100$ ft, PROVIDE GUARDRAIL AND TERMINAL SYSTEM IN ADVANCE OF DRIVE

DRIVE APPLICATION
WITHIN THE CONTROL LINE

Figure 49-9E
<table>
<thead>
<tr>
<th>Median Slope</th>
<th>Design Speed (mph)</th>
<th>Runout Length, $L_R$ (ft)</th>
<th>Clear-Zone Width (ft)</th>
<th>Pay Length (ft) of Double-Faced W-beam Guardrail at 6.25 ft Post Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50</td>
<td>55</td>
<td>60</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>320</td>
<td>360</td>
<td>420</td>
<td>460</td>
</tr>
<tr>
<td>Flatter than 6:1</td>
<td>20.0</td>
<td>23.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td></td>
<td>150.00</td>
<td>181.25</td>
<td>225.00</td>
<td>250.00</td>
</tr>
<tr>
<td>6:1</td>
<td>21.0</td>
<td>26.0</td>
<td>33.0</td>
<td>34.0</td>
</tr>
<tr>
<td></td>
<td>156.25</td>
<td>193.75</td>
<td>237.50</td>
<td>268.75</td>
</tr>
<tr>
<td>5:1</td>
<td>25.0</td>
<td>28.0</td>
<td>36.0</td>
<td>38.0</td>
</tr>
<tr>
<td></td>
<td>168.75</td>
<td>200.00</td>
<td>250.00</td>
<td>275.00</td>
</tr>
</tbody>
</table>

**Note:** The pay length shown in the table is based on the assumed conditions as follows:

1. It is calculated using Section 49-4.02(01), Equation 49-4.3, with $L_2 = 4$ ft.
2. It does not include the guardrail-transition type TGB.
3. W-beam guardrail is flared at 30:1 from the guardrail-transition type TGB to the 12-ft offset, and then is parallel to the roadway.
4. The guardrail-transition type TGB is parallel to and 4 ft from E.T.L.
5. The pay length of bridge-approach guardrail should be recomputed for site conditions other than those assumed above.
6. See the INDOT Standard Drawings.

**MEDIAN BRIDGE-APPROACH CRITERIA**

*Figure 49-9F*
1. FATALITY COST = 500,000
2. SEVERE INJURY COST = 110,000
3. MODERATE INJURY COST = 10,000
4. SLIGHT INJURY COST = 3,000
5. PDO LEVEL 2 COST = 2,500
6. PDO LEVEL 1 COST = 500
7. ENCROACHMENT MODEL = ENC. RATE * (ADTeff ^ ENC. POWER) ENCROACHMENTS/MILE/YR
   = 0.0005000 * (ADTeff ^ 1.000000) ENCROACHMENTS/MILE/YR
8. ENCROACHMENT ANGLE AT 40 MPH = 17.2 DEGREES
9. ENCROACHMENT ANGLE AT 50 MPH = 15.2 DEGREES
10. ENCROACHMENT ANGLE AT 60 MPH = 13.0 DEGREES
11. ENCROACHMENT ANGLE AT 70 MPH = 11.6 DEGREES
12. LIMITING TRAFFIC VOLUME PER LANE = 10,000 VEHICLES PER DAY
13. SWATH WIDTH = 12 FT.

DO YOU WISH TO CHANGE A PARAMETER VALUE (Y/N)?

BASIC DATA INPUT SCREEN
Figure 49-10A
1. TITLE STARTUP VALUES
2. TRAFFIC VOLUME = 0 VPD - TRAFFIC GROWTH = 0.0% PER YEAR
3. DIVIDED ROADWAY 1 ADJACENT LANE(S) OF WIDTH = 12.0 FT.
4. CURVATURE = 0.0 DEGREES GRADE (PERCENTAGE) = 0.0
5. TRAFFIC BASELINE CURVATURE GRADE USER TOTAL VOLUME ENC. FACTOR FACTOR FACTOR ENC.
   ADJACENT 0 0.0000 1.00 1.00 1.00 0.0000
   OPPOSING 0 0.0000 1.00 1.00 1.00 0.0000
6. DESIGN SPEED = 70 MPH ENCROACHMENT ANGLE = 11.6 DEGREES
7. LATERAL (A) = 8 LONGITUDINAL (L) = 200 WIDTH (W) = 1 FT.
8. INITIAL COLLISION FREQUENCY = 0.0000 IMPACTS PER YEAR
   ADJACENT CFT= 0.0000 CF1= 0.0000 CF2= 0.0000 CF3= 0.0000
   OPPOSING CFT= 0.0000 CF4= 0.0000 CF5= 0.0000 CF6= 0.0000
9. SEVERITY INDEX = SU= 0.00 SD= 0.00 CU= 0.00 CD= 0.00 FACE=0.00
   ACCIDENT COST $ 0 $ 0 $ 0 $ 0 $ 0 $ 0 $ 0 $ 0 $ 0 $ 0 $ 0
   KT = 0.962 KJ = 0.962 CRF = 1.040 KC = 0.962
10. PROJECT LIFE = 1 YEARS DISCOUNT RATE = 4.0%
11. INSTALLATION COST = $ 0
12. REPAIR COST/ACC $ $ SU= 0 $ SD= 0 $ CU= 0 $ CD= 0 $ F= 0
13. MAINTENANCE COST /YR = $ 0
14. SALVAGE VALUE = $ 0
15. PRESENT WORTH = $ 0 ANNUALIZED $ 0

HIGHWAY DEPT. COST = $ 0 ANNUALIZED $ 0

INPUT ITEM TO CHANGE (1 TO 14) OR FUNCTION KEY PLUS ENTER
1 PRINT 2 STORE 3 RECALL 4 HELP 5 GLOBAL 6 SI v $ 7 DIR 8 SI DEF 9 GRAPH 10 QUIT

VARIABLE DATA INPUT SCREEN
Figure 49-10B
EDGE OF TRAVEL LANE

EDGE OF SHOULDER

DIRECTION OF TRAVEL

YAW ANGLE

Figure 49-10C
<table>
<thead>
<tr>
<th>Line</th>
<th>Input Data</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Title</td>
<td>49-10.03(01)</td>
</tr>
<tr>
<td>2</td>
<td>Traffic Volume</td>
<td>49-10.03(02)</td>
</tr>
<tr>
<td>3</td>
<td>Roadway Type</td>
<td>49-10.03(03)</td>
</tr>
<tr>
<td>4</td>
<td>Geometric Adjustment Factors</td>
<td>49-10.03(04)</td>
</tr>
<tr>
<td>5</td>
<td>Encroachment Rate</td>
<td>49-10.03(05)</td>
</tr>
<tr>
<td>6</td>
<td>Design Speed</td>
<td>49-10.03(06)</td>
</tr>
<tr>
<td>7</td>
<td>Hazard Definition</td>
<td>49-10.03(07)</td>
</tr>
<tr>
<td>8</td>
<td>Collision Frequency</td>
<td>49-10.03(08)</td>
</tr>
<tr>
<td>9</td>
<td>Severity Index</td>
<td>49-10.03(09)</td>
</tr>
<tr>
<td>10</td>
<td>Project Life and Discount Rate</td>
<td>49-10.03(10)</td>
</tr>
<tr>
<td>11-14</td>
<td>Highway Agency Costs</td>
<td>49-10.03(11)</td>
</tr>
</tbody>
</table>

**INPUT DATA INDEX**

Figure 49-10D
### SUGGESTED LANE DISTRIBUTION

Figure 49-10E

<table>
<thead>
<tr>
<th>AADT</th>
<th>Median Lane</th>
<th>Center Lane</th>
<th>Right Lane</th>
<th>AADT</th>
<th>Median Lane</th>
<th>Right Lane</th>
</tr>
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<tbody>
<tr>
<td>24,000</td>
<td>22</td>
<td>47</td>
<td>31</td>
<td>12,000</td>
<td>20</td>
<td>80</td>
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<td>48,000</td>
<td>31</td>
<td>43</td>
<td>26</td>
<td>24,000</td>
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<tr>
<td>72,000</td>
<td>35</td>
<td>40</td>
<td>25</td>
<td>36,000</td>
<td>33</td>
<td>67</td>
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<td>86,000</td>
<td>37</td>
<td>38</td>
<td>25</td>
<td>48,000</td>
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<td>120,000</td>
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<td>37</td>
<td>26</td>
<td>60,000</td>
<td>50</td>
<td>50</td>
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<tr>
<td>ADT</td>
<td>MEDIAN ANALYSIS</td>
<td>ROADSIDE ANALYSIS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-------</td>
<td>-----------------</td>
<td>-------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>40 mph</td>
<td>50 mph</td>
<td>60 mph</td>
<td>70 mph</td>
<td>40 mph</td>
<td>50 mph</td>
</tr>
<tr>
<td>12,000</td>
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<td>0.47</td>
<td>0.53</td>
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<tr>
<td>48,000</td>
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<td>0.61</td>
<td>0.65</td>
<td>0.68</td>
<td>0.69</td>
<td>0.73</td>
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<tr>
<td>60,000</td>
<td>0.62</td>
<td>0.67</td>
<td>0.70</td>
<td>0.73</td>
<td>0.62</td>
<td>0.67</td>
</tr>
</tbody>
</table>

4-LANE DIVIDED HIGHWAY
USER ADJUSTMENT FACTORS
Figure 49-10F
<table>
<thead>
<tr>
<th>ADT</th>
<th>MEDIAN ANALYSIS</th>
<th>ROADSIDE ANALYSIS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>40 mph</td>
<td>50 mph</td>
</tr>
<tr>
<td>24,000</td>
<td>0.35</td>
<td>0.42</td>
</tr>
<tr>
<td>48,000</td>
<td>0.43</td>
<td>0.49</td>
</tr>
<tr>
<td>72,000</td>
<td>0.46</td>
<td>0.52</td>
</tr>
<tr>
<td>96,000</td>
<td>0.48</td>
<td>0.53</td>
</tr>
<tr>
<td>120,000</td>
<td>0.48</td>
<td>0.53</td>
</tr>
</tbody>
</table>

6-LANE DIVIDED HIGHWAY
USER ADJUSTMENT FACTORS
Figure 49-10G
<table>
<thead>
<tr>
<th>Type of Barrier/Guardrail</th>
<th>Face Side</th>
<th>40 MPH</th>
<th>50 MPH</th>
<th>60 MPH</th>
<th>70 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Both</td>
<td>Range</td>
<td>Avg</td>
<td>Range</td>
<td>Avg</td>
</tr>
<tr>
<td>Strong Post Block-out W-Beam</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Strong Post Block-out Thrie-Beam</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Concrete Safety Shape</td>
<td>Face</td>
<td>1.8 -2.8</td>
<td>2.3</td>
<td>2.4 -3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>Stone Masonry Wall</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Retaining Wall or Vertical Face Barrier</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
</tbody>
</table>

**FACTORS THAT AFFECT SEVERITY RANGE:**

**Low Range:** New installation with proper design, placement and maintenance, area between the travel lane and hardware is flat and free of obstructions, runout area behind hardware clear, recovery area for redirection, adequate soil resistance, proper clearance from obstacle behind barrier, no curb in front/under.

**Mid Range:** Existing installation in fair condition and properly maintained, area between the travel lane and hardware is relatively flat and free of obstructions, no runout area behind hardware, some recovery area for redirection, questionable soil resistance, proper clearance from obstacle behind barrier in most cases, curb under barrier.

**High Range:** Existing installation in questionable condition and/or poorly maintained, height of rail low, bolts or blockouts missing, steep shoulder slope, curb in front, questionable placement with respect to hinge point, inadequate length for tension, high possibility of impact from several directions, insufficient anchorage, improper flare or runout cross-section at terminal, not anchored properly.

**SEVERITY INDICES**
*(Rigid Barrier and Guardrail Parallel to Roadway)*

Figure 49-10H
### FACTORS THAT AFFECT SEVERITY RANGE:

**Low Range:** New installation with proper design, placement and maintenance, area between the travel lane and hardware is flat and free of obstructions, runout area behind hardware clear, recovery area for redirection, adequate soil resistance, proper clearance from obstacle behind barrier, no curb in front/under.

**Mid Range:** Existing installation in fair condition and properly maintained, area between the travel lane and hardware is relatively flat and free of obstructions, no runout area behind hardware, some recovery area for redirection, questionable soil resistance, proper clearance from obstacle behind barrier in most cases, curb under barrier.

**High Range:** Existing installation in questionable condition and/or poorly maintained, height of rail low, bolts or blockouts missing, steep shoulder slope, curb in front, questionable placement with respect to hinge point, inadequate length for tension, high possibility of impact from several directions, insufficient anchorage, improper flare or runout cross-section at terminal, not anchored properly.

### SEVERITY INDICES

*(Guardrail End Treatments)*

**Figure 49-10I**

<table>
<thead>
<tr>
<th>Type of Guardrail End Treatment</th>
<th>Face Side Both</th>
<th>40 MPH</th>
<th>50 MPH</th>
<th>60 MPH</th>
<th>70 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Range</td>
<td>Avg</td>
<td>Range</td>
<td>Avg</td>
</tr>
<tr>
<td>W-Beam Anchored in Backslope</td>
<td>Side</td>
<td>2.4 -3.0</td>
<td>2.7</td>
<td>2.8 -3.6</td>
<td>3.2</td>
</tr>
<tr>
<td>W-Beam Buried End</td>
<td>Side</td>
<td>2.6 -3.2</td>
<td>2.9</td>
<td>3.0 -3.8</td>
<td>3.4</td>
</tr>
<tr>
<td>FHWA Approved Proprietary Guardrail End Treatment</td>
<td>Side</td>
<td>2.2 -2.8</td>
<td>2.5</td>
<td>2.6 -3.2</td>
<td>2.9</td>
</tr>
<tr>
<td>Obsolete/Non-functional</td>
<td>Side</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
</tr>
<tr>
<td>Type of Impact Attenuator</td>
<td>Face Side</td>
<td>40 MPH</td>
<td>50 MPH</td>
<td>60 MPH</td>
<td>70 MPH</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>----------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>Range</td>
<td>Range</td>
<td>Range</td>
<td>Range</td>
</tr>
<tr>
<td>G-R-E-A-T System</td>
<td>Both</td>
<td>2.0 -2.6</td>
<td>2.3</td>
<td>2.4 -3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>Hex-Foam Sandwich System</td>
<td>Both</td>
<td>2.0 -2.6</td>
<td>2.3</td>
<td>2.4 -3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>Gravel Barrels Array</td>
<td>Both</td>
<td>2.0 -2.6</td>
<td>2.3</td>
<td>2.4 -3.0</td>
<td>2.7</td>
</tr>
</tbody>
</table>

**FACTORS THAT AFFECT SEVERITY RANGE:**

*Low Range:* New installation with proper design, placement and maintenance, area between the travel lane and hardware is flat and free of obstructions, runout area behind hardware clear, recovery area for redirection, adequate soil resistance, no curb in front/under.

*Mid Range:* Existing installation in fair condition and properly maintained, area between the travel lane and hardware is relatively flat and free of obstructions, no runout area behind hardware, some recovery area for redirection, questionable soil resistance, curb under barrier.

*High Range:* Existing installation in questionable condition and/or poorly maintained, height of rail low, bolts or blockouts missing, steep shoulder slope, curb in front, questionable placement with respect to hinge point, inadequate length for tension, high possibility of impact from several directions, insufficient anchorage, improper flare or runout cross-section at terminal, not anchored properly.

**SEVERITY INDICES**

(Impact Attenuators)

Figure 49-10J
<table>
<thead>
<tr>
<th>Type and Rate of Parallel Slope</th>
<th>Face Side Both</th>
<th>40 MPH</th>
<th>50 MPH</th>
<th>60 MPH</th>
<th>70 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Range</td>
<td>Avg</td>
<td>Range</td>
<td>Avg</td>
</tr>
<tr>
<td>Foreslope</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10:1 Down</td>
<td>Face</td>
<td>0.2 -0.6</td>
<td>0.4</td>
<td>0.4 -1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>6:1 Down</td>
<td>Face</td>
<td>0.4 -0.8</td>
<td>0.6</td>
<td>0.8 -1.4</td>
<td>1.1</td>
</tr>
<tr>
<td>4:1 Down</td>
<td>Face</td>
<td>1.0 -1.4</td>
<td>1.2</td>
<td>1.4 -2.0</td>
<td>1.7</td>
</tr>
<tr>
<td>3:1 Down</td>
<td>Face</td>
<td>1.6 -2.0</td>
<td>1.8</td>
<td>2.2 -2.8</td>
<td>2.5</td>
</tr>
<tr>
<td>2:1 Down</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>3.2 -3.8</td>
<td>3.5</td>
</tr>
<tr>
<td>Backslope</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4:1 Up</td>
<td>Face</td>
<td>0.6 -1.0</td>
<td>0.8</td>
<td>0.8 -1.4</td>
<td>1.1</td>
</tr>
<tr>
<td>3:1 Up</td>
<td>Face</td>
<td>1.0 -1.4</td>
<td>1.2</td>
<td>1.4 -2.0</td>
<td>1.7</td>
</tr>
<tr>
<td>2:1 Up</td>
<td>Face</td>
<td>1.8 -2.2</td>
<td>2.0</td>
<td>2.2 -2.8</td>
<td>2.5</td>
</tr>
<tr>
<td>Vertical Rock Cut</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Smooth</td>
<td>Face</td>
<td>2.4 -2.8</td>
<td>2.6</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Rough</td>
<td>Face</td>
<td>2.8 -3.2</td>
<td>3.0</td>
<td>3.4 -4.0</td>
<td>3.7</td>
</tr>
</tbody>
</table>

**FACTORS THAT AFFECT SEVERITY RANGE:**

Low Range: Low fill or cut height (0’ to 4’), no objects on slope, traversable (smooth texture such as cut turf or soil), no erosion to trip vehicle, recoverable area within clear zone, rounded hinge points.

Mid Range: Medium fill or cut height (4’ to 8’), objects on slope with less severity (high range) than slope, minor irregular texture (such as uncut or bush-type vegetation, poorly graded soil), minor erosion, recoverable area within clear zone (may be slightly less if consistent through corridor), hinge point with minimum rounding.

High Range: Fill or cut height greater than 8’, objects with approximately same severity within clear zone, rough texture (e.g., rip rap, etc.) hinge point not rounded.

**SEVERITY INDICES**

(Parallel Slopes)

Figure 49-10K
<table>
<thead>
<tr>
<th>Type and Rate of Transverse Slope</th>
<th>Face Side Both</th>
<th>40 MPH</th>
<th>50 MPH</th>
<th>60 MPH</th>
<th>70 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Range</td>
<td>Avg</td>
<td>Range</td>
<td>Avg</td>
</tr>
<tr>
<td>Embankment</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10:1 Up</td>
<td>Side</td>
<td>0.2 -0.6</td>
<td>0.4</td>
<td>0.8 -1.4</td>
<td>1.1</td>
</tr>
<tr>
<td>6:1 Up</td>
<td>Side</td>
<td>1.0 -1.4</td>
<td>1.2</td>
<td>1.4 -2.0</td>
<td>1.7</td>
</tr>
<tr>
<td>4:1 Up</td>
<td>Side</td>
<td>1.8 -2.2</td>
<td>2.0</td>
<td>2.4 -3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>3:1 Up</td>
<td>Side</td>
<td>2.0 -2.4</td>
<td>2.2</td>
<td>2.8 -3.4</td>
<td>3.1</td>
</tr>
<tr>
<td>2:1 Up</td>
<td>Side</td>
<td>3.2 -3.6</td>
<td>3.4</td>
<td>4.0 -4.6</td>
<td>4.3</td>
</tr>
<tr>
<td>Vertical Rock Cut</td>
<td>Side</td>
<td>4.2 -5.0</td>
<td>4.6</td>
<td>5.0 -6.0</td>
<td>5.5</td>
</tr>
</tbody>
</table>

Note: These slopes are upward slopes which transverse the roadway such as at an intersecting driveway, public road approach, or the embankment cone of an overhead bridge.

FACTORs THAT AFFECT SEVERITY RANGE:

Low Range: Height 0 to 4', no objects on slope, traversable (smooth texture such as cut turf or soil), recoverable area within clear zone, rounded hinge points.

Mid Range: Height 4' to 8', objects on slope with less severity (high range) than slope, minor irregular texture (such as uncut or bush type vegetation, poorly graded soil), recoverable area within clear zone (maybe slightly less if consistent through corridor), hinge point with minimum rounding.

High Range: Height greater than 8', objects with approximately same severity within clear zone, rough texture (i.e., rip rap, etc.) hinge point not rounded.

SEVERITY INDICES
(Transverse Slopes)
Figure 49-10L
### Ditch Cross Slope

<table>
<thead>
<tr>
<th>Foreslope</th>
<th>Backslope</th>
<th>Face Side Both</th>
<th>40 MPH Range</th>
<th>40 MPH Avg</th>
<th>50 MPH Range</th>
<th>50 MPH Avg</th>
<th>60 MPH Range</th>
<th>60 MPH Avg</th>
<th>70 MPH Range</th>
<th>70 MPH Avg</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1 Down</td>
<td>3:1 Up</td>
<td>Face</td>
<td>1.8 - 2.4</td>
<td>2.1</td>
<td>2.2 - 3.2</td>
<td>2.7</td>
<td>3.0 - 4.2</td>
<td>3.6</td>
<td>3.6 - 5.0</td>
<td>4.3</td>
</tr>
<tr>
<td>3:1 Down</td>
<td>4:1 Up</td>
<td>Face</td>
<td>1.2 - 1.8</td>
<td>1.5</td>
<td>1.8 - 2.6</td>
<td>2.2</td>
<td>2.4 - 3.6</td>
<td>3.0</td>
<td>2.8 - 4.2</td>
<td>3.5</td>
</tr>
<tr>
<td>3:1 Down</td>
<td>6:1 Up</td>
<td>Face</td>
<td>1.0 - 1.6</td>
<td>1.3</td>
<td>1.4 - 2.2</td>
<td>1.8</td>
<td>2.0 - 3.2</td>
<td>2.6</td>
<td>2.4 - 3.8</td>
<td>3.1</td>
</tr>
<tr>
<td>4:1 Down</td>
<td>3:1 Up</td>
<td>Face</td>
<td>1.2 - 1.8</td>
<td>1.5</td>
<td>1.8 - 2.6</td>
<td>2.2</td>
<td>2.4 - 3.6</td>
<td>3.0</td>
<td>2.8 - 4.2</td>
<td>3.5</td>
</tr>
<tr>
<td>4:1 Down</td>
<td>4:1 Up</td>
<td>Face</td>
<td>1.0 - 1.6</td>
<td>1.3</td>
<td>1.4 - 2.2</td>
<td>1.8</td>
<td>2.0 - 3.2</td>
<td>2.6</td>
<td>2.4 - 3.8</td>
<td>3.1</td>
</tr>
<tr>
<td>4:1 Down</td>
<td>6:1 Up</td>
<td>Face</td>
<td>0.8 - 1.4</td>
<td>1.1</td>
<td>1.2 - 1.8</td>
<td>1.5</td>
<td>1.6 - 2.6</td>
<td>2.1</td>
<td>2.0 - 3.2</td>
<td>2.6</td>
</tr>
<tr>
<td>6:1 Down</td>
<td>3:1 Up</td>
<td>Face</td>
<td>1.0 - 1.6</td>
<td>1.3</td>
<td>1.4 - 2.2</td>
<td>1.8</td>
<td>2.0 - 3.2</td>
<td>2.6</td>
<td>2.4 - 3.8</td>
<td>3.1</td>
</tr>
<tr>
<td>6:1 Down</td>
<td>4:1 Up</td>
<td>Face</td>
<td>0.8 - 1.4</td>
<td>1.1</td>
<td>1.2 - 1.8</td>
<td>1.5</td>
<td>1.6 - 2.6</td>
<td>2.1</td>
<td>2.0 - 3.2</td>
<td>2.6</td>
</tr>
<tr>
<td>6:1 Down</td>
<td>6:1 Up</td>
<td>Face</td>
<td>0.6 - 1.2</td>
<td>0.9</td>
<td>1.0 - 1.6</td>
<td>1.3</td>
<td>1.4 - 2.2</td>
<td>1.8</td>
<td>1.8 - 2.8</td>
<td>2.3</td>
</tr>
</tbody>
</table>

**Note:** For slopes flatter than 6:1 or greater than 3:1, use the appropriate parallel slope values.

**FACTORS THAT AFFECT SEVERITY RANGE:**

*Low Range:* Depth of ditch 0' to 1', flat ditch cross-section (rounded with bottom width > 8', trapezoidal with bottom > 4'), smooth graded surface, rounded hinge points, backslopes clear of objects, no erosion, properly maintained and clear of debris.

*Mid Range:* Depth of ditch 1' to 2.5', objects with approximately the same severity within cross-section, recoverable cross-section between obstacle and traveled way, minor erosion on either slope.

*High Range:* Depth of ditch greater than 2.5', cross-section with abrupt slope changes (Vee ditch, rounded with bottom width < 8', trapezoidal bottom width < 4'), fixed objects on backslopes, steeper cross-section between obstacle and traveled way, erosion on either slope.

**SEVERITY INDICES (Ditches)**

*Figure 49-10M*
<table>
<thead>
<tr>
<th>Culvert and Drainage Item Type and/or Size</th>
<th>Face Side Both</th>
<th>40 MPH Range</th>
<th>Avg</th>
<th>50 MPH Range</th>
<th>Avg</th>
<th>60 MPH Range</th>
<th>Avg</th>
<th>70 MPH Range</th>
<th>Avg</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parallel Slope Culverts</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/End Dia. ≤ 3 ft.</td>
<td>Both</td>
<td>1.6 -2.8</td>
<td>2.2</td>
<td>1.8 -3.2</td>
<td>2.5</td>
<td>2.2 -3.8</td>
<td>3.0</td>
<td>2.6 -4.4</td>
<td>3.5</td>
</tr>
<tr>
<td>w/End Dia. &gt; 3 ft.</td>
<td>Both</td>
<td>2.8 -4.0</td>
<td>3.4</td>
<td>3.2 -4.6</td>
<td>3.9</td>
<td>3.8 -5.4</td>
<td>4.6</td>
<td>4.4 -6.2</td>
<td>5.3</td>
</tr>
<tr>
<td>w/Grated Box End Section</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/Approved Metal</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Safety End Section</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Transverse Slope Culverts</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/End Dia. ≤ 3 ft.</td>
<td>Side</td>
<td>1.8 -3.0</td>
<td>2.4</td>
<td>2.0 -3.4</td>
<td>2.7</td>
<td>2.4 -4.0</td>
<td>3.2</td>
<td>2.8 -4.6</td>
<td>3.7</td>
</tr>
<tr>
<td>w/End Dia. &gt; 3 ft.</td>
<td>Side</td>
<td>3.0 -4.2</td>
<td>3.6</td>
<td>3.4 -4.8</td>
<td>4.1</td>
<td>4.0 -5.6</td>
<td>4.8</td>
<td>4.6 -6.4</td>
<td>5.5</td>
</tr>
<tr>
<td>w/Grated Box End Section</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/Approved Metal</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Safety End Section</td>
<td>Both</td>
<td>&lt;--- USE VALUES FOR APPROPRIATE PARALLEL SLOPE ----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Raised Inlet w/Grate</strong></td>
<td>Both</td>
<td>&lt;--- USE VARIABLE HEIGHT VALUES WITH APPROPRIATE HEIGHT----&gt;</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Rip-rap</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average Dia. &lt; 6&quot;</td>
<td>Both</td>
<td>0.4 -1.0</td>
<td>0.7</td>
<td>1.0 -1.8</td>
<td>1.4</td>
<td>1.4 -2.4</td>
<td>1.9</td>
<td>1.8 -3.0</td>
<td>2.4</td>
</tr>
<tr>
<td>Average Dia. ≥ 6&quot; ≤ 10&quot;</td>
<td>Both</td>
<td>1.0 -2.6</td>
<td>1.8</td>
<td>1.4 -3.2</td>
<td>2.3</td>
<td>1.8 -3.8</td>
<td>2.8</td>
<td>2.2 -4.4</td>
<td>3.3</td>
</tr>
<tr>
<td>Average Dia. &gt; 10&quot;</td>
<td>Both</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
<td>3.8 -7.2</td>
<td>5.5</td>
<td>4.4 -8.6</td>
<td>6.5</td>
</tr>
<tr>
<td><strong>Permanent Stream/Pond</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth ≤ 3 ft.</td>
<td>Both</td>
<td>1.0 -5.0</td>
<td>3.0</td>
<td>1.6 -5.6</td>
<td>3.6</td>
<td>2.2 -6.2</td>
<td>4.2</td>
<td>3.0 -7.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Depth &gt; 3 ft.</td>
<td>Both</td>
<td>5.0 -6.0</td>
<td>5.5</td>
<td>5.6 -6.8</td>
<td>6.2</td>
<td>6.2 -7.6</td>
<td>6.9</td>
<td>7.0 -8.6</td>
<td>7.8</td>
</tr>
</tbody>
</table>

**FACTORS THAT AFFECT SEVERITY RANGE:**

**Low Range:** Flat recoverable area between culvert opening and traveled way, smaller diameter culvert pipe, tapered culvert end section (18" or less), no erosion, properly maintained and clear of debris.

**Mid Range:** Recoverable cross-section between obstacle and traveled way, projecting 24" (or less) culvert end section, minor erosion at opening or inlet.

**High Range:** Steeper cross-section between obstacle and traveled way, projecting culvert end section, large culvert diameter, erosion at opening or inlet.

**SEVERITY INDICES**
(Culverts and Miscellaneous Drainage Items)
Figure 49-10N
<table>
<thead>
<tr>
<th>Type of Object</th>
<th>Face Side Both</th>
<th>40 MPH</th>
<th>50 MPH</th>
<th>60 MPH</th>
<th>70 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range  Avg</td>
<td>Range  Avg</td>
<td>Range  Avg</td>
<td>Range  Avg</td>
<td>Range  Avg</td>
</tr>
<tr>
<td>Utility Pole</td>
<td>Both</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
</tr>
<tr>
<td>Rigid Sign Support</td>
<td>Both</td>
<td>2.2 -4.6</td>
<td>3.4</td>
<td>2.8 -5.6</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
</tr>
<tr>
<td>Breakaway Sign Support</td>
<td>Both</td>
<td>0.6 -1.0</td>
<td>0.8</td>
<td>0.8 -1.4</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>0.8 -1.2</td>
<td>1.0</td>
<td>1.0 -1.6</td>
<td>1.3</td>
</tr>
<tr>
<td>Luminaire Support</td>
<td>Both</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>2.0 -2.4</td>
<td>2.2</td>
<td>2.2 -2.8</td>
<td>2.5</td>
</tr>
<tr>
<td>Pedestal or Foundation</td>
<td>Both</td>
<td>0.6 -1.0</td>
<td>0.8</td>
<td>1.0 -1.8</td>
<td>1.4</td>
</tr>
<tr>
<td>Height &lt; 4&quot;</td>
<td>Both</td>
<td>1.0 -2.6</td>
<td>1.8</td>
<td>1.8 -3.2</td>
<td>2.5</td>
</tr>
<tr>
<td>Height ≥ 4&quot; ≤ 10&quot;</td>
<td>Both</td>
<td>2.6 -5.0</td>
<td>3.8</td>
<td>3.2 -6.0</td>
<td>4.6</td>
</tr>
</tbody>
</table>

Note: The surrounding slope may be more severe than the object on it. Additionally, another hazard directly beyond the object may be more severe. In most cases, the highest severity index should be used.

**FACTORS THAT AFFECT SEVERITY RANGE:**

Low Range: Object on uphill backslope where less likely to be hit, non-frangible diameter is small, new installation of frangible object with clear recovery area behind, top of base flush with ground, no erosion around base.

Mid Range: Object on relatively flat slope (recoverable), new installation of sign support on relatively flat slope, proper design, installation and maintenance, top of base less than 4 inches above ground, no erosion around base.

High Range: Object on 4:1 or steeper slope (non-recoverable), non-frangible diameter is large, improper placement and/or design of post, base located at hinge, erosion around base, improper maintenance of sign support and breakaway device.

**SEVERITY INDICES**

(Utility, Sign, and Luminaire Support Fixed Objects)
Note: The surrounding slope may be more severe than the object, and another hazard directly beyond the object may be more severe. In most cases, the highest severity index should be used.

**FACTORS THAT AFFECT SEVERITY RANGE:**

**Low Range:** Object on uphill backslope where less likely to be hit, non-frangible diameter is small, no erosion around base.

**Mid Range:** Object on relatively flat slope (recoverable), proper design, installation and maintenance, no erosion around base.

**High Range:** Object on 4:1 or steeper slope (non-recoverable), non-frangible diameter is large, erosion around base.

---

### SEVERITY INDICES

*(Miscellaneous Fixed Objects)*

*Figure 49-10P*
<table>
<thead>
<tr>
<th>TYPE OF BARRIER/TERMINAL</th>
<th>REPAIR COST PER ACCIDENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Beam Guardrail</td>
<td>$ 1200.00</td>
</tr>
<tr>
<td>Concrete Safety Shape</td>
<td>$ 0.00</td>
</tr>
<tr>
<td>Buried End</td>
<td>$ 500.00</td>
</tr>
<tr>
<td>C-A-T Unit</td>
<td>$ 4000.00</td>
</tr>
<tr>
<td>SENTRE System</td>
<td>$ 4000.00</td>
</tr>
<tr>
<td>ET 2000</td>
<td>$ 4000.00</td>
</tr>
<tr>
<td>BRAKEMASTER System</td>
<td>$ 4000.00</td>
</tr>
<tr>
<td>G.R.E.A.T. System</td>
<td>$ 4000.00</td>
</tr>
<tr>
<td>Gravel Barrel Array</td>
<td>$ 3000.00</td>
</tr>
<tr>
<td>Hex-Foam Sandwich System</td>
<td>$ 4000.00</td>
</tr>
</tbody>
</table>

REPAIR COSTS
Figure 49-10Q
<table>
<thead>
<tr>
<th>INPUT VEHICLE</th>
<th>USER COST</th>
<th>AGENCY COST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic Volume</td>
<td>Significant</td>
<td>Minor</td>
</tr>
<tr>
<td>Traffic Growth</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Curvature/Grade</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Design Speed</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Lateral Placement</td>
<td>Significant</td>
<td>Minor</td>
</tr>
<tr>
<td>Longitudinal Length</td>
<td>Significant</td>
<td>Significant</td>
</tr>
<tr>
<td>Width</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Severity Index</td>
<td>Significant</td>
<td>N/A</td>
</tr>
<tr>
<td>Project Life</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Interest Rate</td>
<td>Minor</td>
<td>Minor</td>
</tr>
<tr>
<td>Installation Cost</td>
<td>N/A</td>
<td>Significant</td>
</tr>
<tr>
<td>Repair Cost</td>
<td>N/A</td>
<td>Minor</td>
</tr>
<tr>
<td>Accident Cost</td>
<td>Significant</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**SUMMARY OF INPUT VARIABLE RELATIVE SIGNIFICANCE**

*Figure 49-10R*
<table>
<thead>
<tr>
<th>GRET Type</th>
<th>ADT</th>
<th>Clearances (ft)</th>
<th>Severity Index&lt;sup&gt;(2)&lt;/sup&gt;</th>
<th>Installation Costs&lt;sup&gt;(3)&lt;/sup&gt; ($)</th>
<th>Repair Costs&lt;sup&gt;(4)&lt;/sup&gt; ($)</th>
<th>Guardrail Present Worth ($)</th>
<th>Equivalent Embankment SI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20-yr</td>
<td>Current</td>
<td>Lat</td>
<td>Long.&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>Side</td>
<td>Face</td>
<td>Side</td>
</tr>
<tr>
<td>I</td>
<td>700</td>
<td>471</td>
<td>10</td>
<td>1300</td>
<td>2.8</td>
<td>2.5</td>
<td>13,900</td>
</tr>
<tr>
<td>I</td>
<td>1000</td>
<td>673</td>
<td>10</td>
<td>1350</td>
<td>2.8</td>
<td>2.5</td>
<td>14,400</td>
</tr>
<tr>
<td>I</td>
<td>1500</td>
<td>1009</td>
<td>10</td>
<td>1350</td>
<td>2.8</td>
<td>2.5</td>
<td>14,400</td>
</tr>
<tr>
<td>I</td>
<td>2000</td>
<td>1346</td>
<td>12</td>
<td>1350</td>
<td>2.8</td>
<td>2.5</td>
<td>14,400</td>
</tr>
<tr>
<td>I</td>
<td>3000</td>
<td>2019</td>
<td>12</td>
<td>1380</td>
<td>2.8</td>
<td>2.5</td>
<td>14,700</td>
</tr>
<tr>
<td>I</td>
<td>6000</td>
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<td>12</td>
<td>1380</td>
<td>2.8</td>
<td>2.5</td>
<td>14,700</td>
</tr>
<tr>
<td>I</td>
<td>9000</td>
<td>6057</td>
<td>12</td>
<td>1450</td>
<td>2.8</td>
<td>2.5</td>
<td>15,400</td>
</tr>
<tr>
<td>I</td>
<td>12000</td>
<td>8076</td>
<td>12</td>
<td>1450</td>
<td>2.8</td>
<td>2.5</td>
<td>15,400</td>
</tr>
<tr>
<td>I</td>
<td>18000</td>
<td>12113</td>
<td>12</td>
<td>1450</td>
<td>2.8</td>
<td>2.5</td>
<td>15,400</td>
</tr>
</tbody>
</table>

Notes:

1. Guardrail = 1000 ft + 2L<sub>R</sub>, where L<sub>R</sub> is from Figure 49-5F. Embankment = 1000 ft.
2. Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.
3. Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I. If 2 GRET type OS or MS used instead, cost increases by $6100.
4. Repair Costs for side based on use of GRET type I. If GRET type OS or MS used instead, such cost becomes $4000.
<table>
<thead>
<tr>
<th>GRET Type</th>
<th>ADT</th>
<th>Clearances (ft)</th>
<th>Severity Index (2)</th>
<th>Installation Costs ($)</th>
<th>Repair Costs ($) (4)</th>
<th>Guardrail Present Worth ($)</th>
<th>Equivalent Embankment SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>700</td>
<td>471</td>
<td>10 1380</td>
<td>3.1 2.8</td>
<td>14,700</td>
<td>500 1200</td>
<td>14,900 5.279</td>
</tr>
<tr>
<td>I</td>
<td>1000</td>
<td>673</td>
<td>10 1410</td>
<td>3.1 2.8</td>
<td>14,700</td>
<td>500 1200</td>
<td>15,900 4.946</td>
</tr>
<tr>
<td>I</td>
<td>1500</td>
<td>1009</td>
<td>10 1410</td>
<td>3.1 2.8</td>
<td>15,000</td>
<td>500 1200</td>
<td>17,000 4.445</td>
</tr>
<tr>
<td>I</td>
<td>2000</td>
<td>1346</td>
<td>12 1480</td>
<td>3.1 2.8</td>
<td>15,700</td>
<td>500 1200</td>
<td>17,600 4.311</td>
</tr>
<tr>
<td>I</td>
<td>3000</td>
<td>2019</td>
<td>12 1480</td>
<td>3.1 2.8</td>
<td>15,700</td>
<td>500 1200</td>
<td>20,400 4.058</td>
</tr>
<tr>
<td>I</td>
<td>6000</td>
<td>4038</td>
<td>12 1510</td>
<td>3.1 2.8</td>
<td>16,000</td>
<td>500 1200</td>
<td>26,700 3.494</td>
</tr>
<tr>
<td>I</td>
<td>9000</td>
<td>6057</td>
<td>12 1510</td>
<td>3.1 2.8</td>
<td>16,000</td>
<td>500 1200</td>
<td>33,600 3.306</td>
</tr>
<tr>
<td>I</td>
<td>12000</td>
<td>8076</td>
<td>12 1510</td>
<td>3.1 2.8</td>
<td>16,000</td>
<td>500 1200</td>
<td>40,000 3.199</td>
</tr>
<tr>
<td>I</td>
<td>18000</td>
<td>12113</td>
<td>12 1450</td>
<td>3.1 2.8</td>
<td>15,400</td>
<td>500 1200</td>
<td>52,800 3.093</td>
</tr>
</tbody>
</table>

Notes:

(1) Guardrail = 1000 ft + 2L_R, where L_R is from Figure 49-5F. Embankment = 1000 ft.

(2) Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.

(3) Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I. If 2 GRET type OS or MS used instead, cost increases by $6100.

(4) Repair Costs for side based on use of GRET type I. If GRET type OS or MS used instead, such cost becomes $4000.

**BACKUP DATA FOR FIGURE 49-3B(45)**

*Figure 49-11B*
<table>
<thead>
<tr>
<th>GRET Type</th>
<th>ADT</th>
<th>Clearances (ft)</th>
<th>Severity Index(2)</th>
<th>Installation Costs ($)</th>
<th>Repair Costs ($) (4)</th>
<th>Guardrail Present Worth ($)</th>
<th>Equivalent Embankment SI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20-yr</td>
<td>Current</td>
<td>Lat</td>
<td>Long.(1)</td>
<td>Side</td>
<td>Face</td>
<td>Side</td>
</tr>
<tr>
<td>I</td>
<td>700</td>
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</table>

Notes:

(1) Guardrail = 1000 ft + 2L_R, where L_R is from Figure 49-5F. Embankment = 1000 ft.

(2) Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.

(3) Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I. If 2 GRET type OS or MS used instead, cost increases by $6100.

(4) Repair Costs for side based on use of GRET type I. If GRET type OS or MS used instead, such cost becomes $4000.

**BACKUP DATA FOR FIGURE 49-3B(50)**

Figure 49-11C
<table>
<thead>
<tr>
<th>GRET Type</th>
<th>AADT</th>
<th>Clearances (ft)</th>
<th>Severity Index (2)</th>
<th>Installation Costs ($)</th>
<th>Repair Costs ($) (4)</th>
<th>Guardrail Present Worth ($)</th>
<th>Equivalent Embankment SI</th>
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<td>Current 471</td>
<td>Lat 10 Long.(1)</td>
<td>Side 3.8 Face 3.4</td>
<td>16,300 Side 500 Face 1200</td>
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<tr>
<td>I</td>
<td>20-yr 1000</td>
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<tr>
<td>OS or MS</td>
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<td>Lat 12 Long.(1)</td>
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<tr>
<td>OS or MS</td>
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<td>24,100 Side 4000 Face 1200</td>
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<td>3.681</td>
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Notes:

1. Guardrail = 1000 ft + 2L_R, where L_R is from Figure 49-5F. Embankment = 1000 ft.

2. Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.

3. Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I, or based on $10/ft for guardrail + $7000 for 2 GRET type OS or MS.

4. Repair Cost of $500 for side based on use of GRET type I. Repair Cost of $4000 for side based on use of GRET type OS or MS.

Backup Data for Figure 49-4E(55)
<table>
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<th>End Treatment</th>
<th>AADT</th>
<th>Clearances (ft)</th>
<th>Severity Index (1)</th>
<th>Installation Costs ($)</th>
<th>Repair Costs ($)</th>
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<tr>
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<td>471</td>
<td>10 1610</td>
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<tr>
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Notes:

1. Guardrail = 1000 ft + 2L_R, where L_R is from Figure 49-5F. Embankment = 1000 ft

2. Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.

3. Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I, or based on $10/ft for guardrail + $7000 for 2 GRET type OS or MS.

4. Repair Cost of $500 for side based on use of GRET type I. Repair Cost of $4000 for side based on use of GRET type OS or MS.

**BACKUP DATA FOR FIGURE 49-3B(60)**

**Figure 49-11E**
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<th>Installation Costs ($)</th>
<th>Repair Costs ($)</th>
<th>Guardrail Present Worth ($)</th>
<th>Equivalent Embankment SI</th>
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<td>Face</td>
<td>Side</td>
<td>Face</td>
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<td>673</td>
<td>10 1770</td>
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<td>1009</td>
<td>10 1770</td>
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Notes:

1. Guardrail = 1000 ft + 2\(L_R\), where \(L_R\) is from Figure 49-5F. Embankment = 1000 ft.

2. Severity Index interpolated for design speed from Figures 49-10H and 49-10 I.

3. Cost shown is based on $10/ft for guardrail + $900 for 2 GRET type I, or based on $10/ft for guardrail + $7000 for 2 GRET type OS or MS.

4. Repair Cost of $500 for side based on use of GRET type I. Repair Cost of $4000 for side based on use of GRET type OS or MS.

**BACKUP DATA FOR FIGURE 49-3B(65)**
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* Values were interpolated.
** Values were estimated graphically.

**SEVERITY INDICES**

Figure 49-11G
CHAPTER 304

Comprehensive Pavement Analyses

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<td>Performance Criteria for New or Rehabilitation HMA Pavement</td>
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CHAPTER 304

COMPREHENSIVE PAVEMENT ANALYSES

304-1.0 INTRODUCTION

This chapter provides guidance for the investigation, evaluation, and analyses of pavements for the public roadway system in Indiana. The pavement analyses shall be based on, but not limited to, sound pavement engineering principles, concepts, and economics, as well as geotechnical conditions, environmental conditions, pavement material properties, and traffic loadings.

304-2.0 HISTORY

The history of pavements in Indiana has transcended a number of types and configurations from surfaces using bricks, aggregates, and Kentucky rock asphalt, to the most modern Superpave Asphalt Binders. Kentucky rock asphalt is naturally occurring asphalt that has not been used in recent years but can be found within an existing pavement structure when coring the roadway. Sand surfaces were used extensively on asphalt pavements in the 1970’s and 1980’s. Both sand surfaces and Kentucky rock asphalts appear as a relatively thin black dense layer in the core, typically less than an inch thick.

Most of the initial interstate pavement constructed in the 1960’s and early 1970’s was either continuously reinforce concrete (CRC) or jointed reinforced concrete pavement (JRCP) with 40-ft joint spacing. In the early 1980’s these concrete pavements were undersealed and overlaid with at least two lifts of HMA as a first rehabilitation measure. In the 1990’s the HMA was milled or removed and new HMA applied as a rehabilitation measure. Also in the 1990’s the HMA was removed and the concrete pavements on the interstates were either cracked and seated, or rubblized as a new method of slab reduction emerged, these concrete pavements were then resurfaced with at least two lifts of HMA. INDOT did not get good performance from the cracked and seated pavements as the technology in equipment used to crack the concrete had not advanced enough.

The National Highway System (NHS) routes were also constructed with different typical cross-sections; such as variable thickness 9”-7”-9” from edge to center to edge with portland cement concrete. These NHS routes were also typically 18’-20’ wide. Tilt sections were also common in the early interstate and NHS pavements. As the tilt section pavements reached the point of rehabilitation INDOT converted them to crown sections by milling and applying variable thickness of HMA overlays.
Pavements on most state routes were initially 9-ft lanes, with little to no shoulders. Some of these routes were initially county roads that were given to the State. Asphalt pavements used sand surfaces, hot asphalt emulsions (HAE), Bituminous coated aggregate (BCA) or Greasy 5’s on these routes in the early days. The majority of all pavements today have been widened to at least 10-ft, 11-ft, or 12-ft lanes, with or without shoulders depending on the available right-of-way. Beginning in about 1992, SuperPave Performance Grade (PG) binders were being used and replaced the old Marshall Method of HMA binder design. Beginning in 2011 all new HMA pavement applied to these state routes with aggregate or earth shoulders had the safety edge incorporated.

Underdrains have been utilized since the 1950’s. Transverse underdrains were some of the first underdrains installed. Beginning in the 1960’s, longitudinal pipes were constructed along the edges of the pavement and outlet to the side ditches. Geocomposite edge drains were used as retrofit underdrains from the mid 1980’s to the mid 1990’s. From the mid 1990’s to present retrofit underdrains consist of open graded material and 4-in. pipe along the pavement’s edge. Little or no maintenance has been performed on the underdrain systems and studies show that poor performance of the underdrain systems is a primary cause of failures of pavement structures. INDOT has also improved on the design of underdrain systems since the mid 1990’s to facilitate better maintenance. This includes 45° elbows to facilitate video logging, paved outlet protector pads, and rodent screens. INDOT district maintenance now has underdrain maintenance as an activity on the Work Management System (WMS).

304-3.0 ABBREVIATIONS

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>AADT</td>
<td>Average Annual Daily Traffic</td>
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<tr>
<td>AADTT</td>
<td>Average Annual Daily Truck Traffic</td>
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
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<td>AC</td>
<td>Asphaltic Concrete</td>
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<td>ACPA</td>
<td>American Concrete Pavement Association</td>
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<tr>
<td>ADA</td>
<td>Americans with Disabilities Act</td>
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<td>APAI</td>
<td>Asphalt Pavement Association of Indiana</td>
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<td>ASR</td>
<td>Alkali-Silica Reactivity</td>
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<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
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<tr>
<td>CAB</td>
<td>Compacted Aggregate Base</td>
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<td>CAP</td>
<td>Compacted Aggregate Pavement</td>
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<td>CBR</td>
<td>California Bearing Ratio</td>
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<td>CCPR</td>
<td>Cold Central Plant Recycling</td>
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<tr>
<td>CIR</td>
<td>Cold In-Place Recycling</td>
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<td>CPR</td>
<td>Concrete-Pavement Restoration</td>
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304-4.0 INDOT PAVEMENT ANALYSES PHILOSOPHY

INDOT pavement analyses and design philosophy are based on the least cost of ownership, represented by cost/lane mile/year of life. INDOT pavements should be investigated, evaluated, analyzed, and designed to cost the least amount of money over the design life of the treatment, and constructed using Quality Control/Quality Assurance (QC/QA) materials to be durable and be structurally and functionally sound through that entire period. This pavement design process includes, but is not limited to:

1. Investigation
   a. History, age
   b. Falling Weight Deflectometer (FWD)
   c. Coring
   d. Geotechnical
   e. Pavement data
   f. Traffic data
2. Evaluation
   a. Identify types of distresses
   b. Causes of distresses
   c. Functional versus structural distress
3. Analyses
   a. Alternate pavement treatments
   b. Mechanistic-Empirical Pavement Design Guide (MEPDG), AASHTOWare Pavement ME Design Software

d. Maintenance considerations

304-5.0 PAVEMENT DESIGN DEVELOPMENT

The pavement design process should be a continuous development flow as data is collected and alternatives are considered. However, there are milestones to be considered during the process. The milestones include Preliminary Pavement Proposal (0%-30% of overall project development), Final Pavement Design (by 60% of overall project development), and Pavement Design Validation (by 90% of overall project development).

The Pavement Design Engineer will recommend the pavement type and thickness of the pavement layers of the pavement structure based on subgrade conditions, materials, traffic, environment, economic, and other considerations.

A Pavement Design Engineer is a qualified licensed engineer who has been trained in pavement design analysis. Throughout this chapter the Pavement Design Engineer will be referred to as the pavement designer. A pavement designer may be a consultant, a district Pavement Engineer, or Central Office Pavement Engineer. For consultant pavement designers, there is a required pre-qualification process, which includes certain prerequisite courses. Courses are available through the National Highway Institute (NHI) and listed on the Pavement Engineering Section of the Standards and Specifications webpage, http://www.in.gov/dot/div/contracts/standards/. Comparable university or Department-led courses may be substituted for NHI courses with the approval of the Pavement Division director. The Pavement Division Office of Pavement Engineering should be contacted to initiate the process. A pavement designer is responsible for the following:

1. Identification of the extent and severity of distresses,

2. Selection of proposed pavement treatment alternatives based on these distresses,

3. Collection of pavement history

4. Determination of estimated cost of proposed pavement treatment alternatives,

5. Requests that other data be obtained, including the following:
   a. Falling Weight Deflectometer (FWD)
b. Cores  
c. Geotechnical  
d. Traffic data from appropriate source, with % truck  
e. Ground Penetrating Radar (GPR)

6. Analyses  
a. At a minimum, one optimal design and one failure iteration in accordance with the Mechanistic-Empirical Pavement Design Guide (MEPDG) methodology utilizing AASHTOWare Pavement ME Design software  
b. Life-Cycle Cost Analysis (LCCA)  
c. Alternate Pavement Types Determination  
   i. HMA  
   ii. PCCP  
d. Specifying HMA mixture properties including ESAL, PG, course and mix designation of a project

7. Maintenance Considerations

8. Pavement Design Validation by 60%-90% overall project development

9. Pavement Design Life

304-5.01 INDOT Pavement Design Process

Every INDOT proposed pavement project must be evaluated for proper treatment prior to being added to a construction and funding program as a project in the Call for Projects. The project intent and its impacts on the existing or new pavement structure should be understood prior to developing the pavement treatment recommendation.

Pavement replacement over culvert/pipe replacement or utility projects that result in small cuts of no more than 100’ wide or long, shall match the type and thickness of the existing pavement and may not require a computer iteration; however, the pavement designer shall check the structural adequacy of the existing pavement to carry the current and future projected traffic loads. This may also include small projects to address isolated rutting issues in a single lane or ramp only issues. This minimal pavement design will include pavement history or cores, pavement condition assessment, and appropriate drainage and subsurface drainage provisions. The pavement designer will specify HMA or PCCP thickness, HMA mixture designation (based on AADTT), and a
minimum subgrade treatment requirement. Use Figure 304-15B for HMA mixture designations. See INDOT Standard Specifications Section 400 or 500 for Pay Items.

304-5.01(01) INDOT Preliminary Pavement Proposal

The project intent is not always driven by the pavement design, e.g., improved safety, addition of travel lanes, interchange construction, improved sight distance, ADA compliance, correction of deficient drainage, or correction of geometric deficiencies. The project manager must communicate and collaborate with the pavement designer to determine and overcome critical project challenges such as maintenance of traffic (MOT), pavement widening, over-all geometrics, temporary pavements, pavement patching, or drainage.

Pavement distresses are the first characteristics that should be determined and described in consideration of the appropriate treatment for the project. See the Distress Identification Manual for the Long-Term Pavement Performance Program, Publication Number: FHWA-RD-03-031, latest edition for additional information.

The preliminary proposed pavement treatment or scope will identify pavement alternatives to correct pavement structural or functional problems at the start of a project. The preliminary pavement scope for INDOT projects will come from the data produced from Department’s Pavement Management System, such as International Roughness Index (IRI), rut depth, Friction Number (FN), cracking, as well as any additional data that is available. It may not include FWD data, cores and geotechnical investigation.

The preliminary pavement scope will fall into one of the following four project categories:

1. New Alignment,
2. Pavement Reconstruction,
3. Pavement Rehabilitation
   a. Structural Overlay (PCCP > 4 in. or 2 or more HMA layers (surface and intermediate)
   b. PCCP Rubblization and HMA Overlay
   c. PCCP Cracking and Seating and HMA Overlay
   d. Unbonded PCCP overlay over old PCCP
   e. Full Depth Reclamation
4. Preventive Maintenance
   a. Surface Treatment
b. HMA Mill and Fill Overlay  
c. In-Place Recycling  
d. Pavement Preservation Initiative (PPI)  
e. Crack Seal/Fill  

Each category has numerous alternative treatments to be considered to accomplish the intent of the project. Added travel lanes projects may be included in Pavement Reconstruction, Pavement Rehabilitation or Preventive Maintenance (Mill and Fill) Projects.

The pavement designer in collaboration with Pavement Area Engineer shall submit a preliminary pavement proposal for review at 0-30% of overall project development to the Director of Pavement Division. The preliminary pavement proposal scope should consist of the following:

1. clear identification of pavement type, extent, and severity of distresses;  
2. core report, if available, with core photographs to determine pavement structure;  
3. site visit findings and recommendations with photographs;  
4. other pavement history and data, such as original construction and all rehabilitations;  
5. selected proposed pavement treatment alternatives based on distresses;  
6. structural capacity of the pavement treatment alternatives based on initial AASHTOWare Pavement ME software iterations;  
7. determination of estimated cost of proposed pavement treatment alternatives; and  
8. traffic data, with % truck.  

The preliminary pavement proposal shall state what subsequent additional activities or testing must be obtained, i.e., a geotechnical investigation, FWD data, cores, projected construction year traffic data, and/or other testing data. The subsequent activities should be as appropriate to further identify the causes of distress and obtain necessary data to help select the appropriate alternative and to finalize the design to achieve the most effective solution at least cost of ownership.

304-5.01(02) INDOT Final Pavement Design

Final pavement design should by the 60% overall project development. Consideration for the use of underdrains in the pavement section must be in accordance with Section 304-18.0.

Pavement design completed by the district or central office pavement designer should be routed through the Central Office Area Pavement Engineer, Pavement Engineering Manager, and Pavement Division Director for review and approval. The approved final pavement design is then returned to the designer.
This process assures that all pavement designs are checked by a qualified peer. The final pavement design shall be signed, dated, and sealed with an active Professional Engineering stamp by the pavement designer responsible for the design.

The final pavement design memorandum should include the intent of the project, existing pavement type, history of the pavement from initial construction through the last treatment, thickness of all layers, site visit findings and recommendations, testing data findings and recommendations, table of design data, pavement design recommendations, patching summary table, and other pertinent information like utilities or constructability issues. Constructability issues may include temporary widening, temporary runarounds, temporary ramps, rubblization, and other challenges. A patching summary table definitely needs to be finalized by the district Pavement Engineer and the design engineer and included in the contract documents before the letting.

A consultant pavement designer contracted by INDOT shall submit the final pavement design by memorandum on their letterhead including a report with the following information. The submittal shall provide evidence that all pavement designs are checked and signed by a qualified peer.

1. Executive Summary;
2. Project Description;
3. Pavement History;
4. Methodology for selecting preferred pavement strategy;
5. Assessment of Current Pavement Condition with photographs;
6. Pavement Design and Recommendations, including no less than three Alternate Pavement types;
7. Life Cycle Cost Analysis (LCCA) for projects equal to or greater than 10,000 yd²;
8. Construction and Maintenance; and
9. Appendices as follows:
   a. Traffic Data;
   b. Geotechnical Investigation Report;
   c. Pavement Cores with Photographs;
   d. Non-Destructive Testing Results, such as FWD;
   e. HMA Binder Selection using LTPPBind;
   f. Typical Sections;
   g. AASHTOWare Pavement ME Design input Summary;
   h. AASHTOWare Pavement ME Design output, at least the optimal design and then one failure iteration; and
   i. LCCA Results.
The qualifications of the pavement designer noted in Section 304-5.0 apply to LPA projects. The project intent and its impacts on the pavement structure should be understood prior to developing the pavement treatment recommendation. LPA pavement designs will be reviewed and approved by INDOT as noted in the following sections.

304-5.02(01) LPA Final Pavement Design for Locally-Owned, Non-NHS Routes [Rev. Feb. 2018]
Projects that include work on a locally-owned, non-NHS route do not require review and approval by INDOT.
The LPA may follow the INDOT pavement design process (Section 304-5.01), or choose to use their own pavement design criteria.

Where an LPA chooses to use their own pavement design criteria, the following will apply:

- The LPA is responsible for the design and performance of the pavement section.
- A life-cycle cost analysis in accordance with Section 304-20.0 is not required.
- It is the LPA’s responsibility to ensure that the pavement pay items are compatible with the INDOT Standard Specifications.
- If the LPA uses a standard typical pavement section, it must be included in the final pavement design report.
- The final pavement design report must be initialed by the Employee in Responsible Charge (ERC) sealed, signed, and dated by a licensed Indiana Professional Engineer and uploaded to ERMS as the Final Pavement Design.

304-5.02(02) LPA Final Pavement Design for State and NHS Routes [Rev. Feb. 2018]
Projects that include work on a State route or NHS route must be reviewed and approved by a Central Office pavement design engineer and follow the INDOT pavement design process. See Section 304-5.01.
Standard pavement sections may be used in lieu of project-specific pavement designs for low volume roads and trails as follows:

1. Aggregate Pavement on Low Volume Roads, AADTT ≤ 50 trucks. The pavement section will consist of:

   4” Compacted Aggregate No. 73, on
   6” Compacted Aggregate No. 53, on
   Subgrade Treatment, Type III, or as specified in the geotechnical report

   See Typical Section on Figure 304-21AA.

2. Trails and other Non-Vehicular Use Facilities. The pavement sections will consist of the section as shown on the INDOT Standard Drawings series 502-NVUF for concrete pavement and 604-NVUF for HMA pavement.

304-5.02(04) Notification of Pavement Design Approval [Rev. Feb. 2018]
For projects reviewed and approved by INDOT, the Central Office pavement engineer will send a Letter of Pavement Analysis/Design Acceptance (acceptance letter) to the ERC, INDOT project manager, and the LPA pavement designer.

The acceptance letter should be initialed by the ERC, combined with the final pavement design report, and uploaded into ERMS as the Final Pavement Design. Preferably, the file should be upload within two weeks of receiving the acceptance letter. The pavement designer should notify the district coordinator, INDOT project manager, INDOT Central Office pavement design coordinator, and the ERC when the file has been uploaded.

304-6.0 PAVEMENT PROJECT CATEGORIES

INDOT pavement projects will fall in one of the following four project categories:

1. New Alignment;

2. Pavement Reconstruction;

3. Pavement Rehabilitation; or
a. Structural Overlay
b. PCCP Rubblization and HMA Overlay
c. PCCP Cracking and Seating and HMA Overlay
d. Unbonded PCCP overlay over old PCCP
e. Full Depth Reclamation

4. Preventive Maintenance.
   a. Surface Treatment
   b. HMA Mill and Fill
c. In-Place Recycling Technologies

The pavement should be designed in accordance with Section 304-14.0, MEPDG using AASHTOWare Pavement ME Design software, formerly DARWin ME.

**304-6.01 New Alignment [Rev. Mar. 2018]**

New Alignment projects include pavement designs that include recommendations for preparation of the subgrade prior to placing the new pavement structure. Recommendations for New Alignment projects typically include a pavement thickness for both asphalt and concrete pavement.

**304-6.02 Pavement Reconstruction [Rev. Mar. 2018]**

Pavement reconstruction is defined as the replacement or reestablishment of the original pavement structural capacity by the placement of the equivalent or increased pavement structure on the existing alignment. Pavement replacement projects include removal of the existing pavement structure, including subbase, and preparation of the foundation soil and subgrade prior to placing a new pavement structure. Pavement damaged due to structural deficiencies should be considered for replacement. Pavement reconstruction may utilize either new or recycled materials for the reconstruction of the complete pavement structure.

**304-6.03 Pavement Rehabilitation [Rev. Mar. 2018]**

Pavement Rehabilitation is defined as work consisting of structural enhancements that extend the life of an existing pavement and/or improve its structural capacity. A widening component may be included with a rehabilitation or structural overlay project. Rehabilitation techniques include restoration treatments and/or structural overlays. A pavement that is currently structurally
insufficient or will be insufficient based on future traffic is a candidate for a rehabilitation type project.

304-6.03(01) Structural Overlay

The majority of Pavement Rehabilitation projects add pavement structure with an overlay. This may include partial recycling of the existing pavement, placement of additional surface materials, and/or other work necessary to return an existing pavement to a condition of structural adequacy. A pavement structural overlay will be by design, but may generally be:

1. 2 layer HMA overlay, or thin PCCP (4”–6”), also known as minor structural treatment; or
2. ≥ 3 layers of HMA overlay, or PCCP ≥ 6”, also known as major structural treatment.

304-6.03(02) PCCP Rubblization and HMA Overlay

An effective way to rehabilitate a PCCP that has lost structural capacity is to rubblize the existing PCCP and overlay with HMA (2 or 3 layers by design). Rubblizing consists of breaking the concrete into particles ranging from sand size to pieces not exceeding 6 in. in the largest dimension, with the majority being a nominal 1 to 2 in. in size. The concrete from the surface to the top of the reinforcements shall be reduced to the 1 to 2 in. size to the fullest extent possible. INDOT Standard Specifications, Section 305.04(d).

Underdrains shall be designed and placed along the edges of the pavement prior to rubblization.

Prior to placing the HMA overlay, the complete width of the rubblized pavement shall be compacted by means of vibratory steel wheel and pneumatic-tired rollers in accordance with 409.03(b). A prime coat shall be applied after rolling and before overlay, see section 304-17.08.

Traffic will not be allowed on the rubblized pavement before the HMA base or intermediate courses are placed. The initial HMA course shall be placed within 48 hours of rubblizing. In the event of rain prior to placing the overlay, the 48 hour time limitation shall be waived to allow sufficient time for the rubblized pavement to dry.
304-6.03(03) PCCP Cracking and Seating and HMA Overlay

Another effective way to rehabilitate a PCCP that has lost structural capacity is to crack and seat the existing PCCP and overlay with HMA (2 or 3 layers by design). Cracking and seating consists of cracking the existing PCCP pavement and requires a unique special provision.

Underdrains shall be designed and placed along the edges of the pavement prior to crack-and-seat.

Prior to placing the HMA overlay, the complete width of the cracking and seated pavement shall be compacted by means of vibratory steel wheel and pneumatic-tired rollers in accordance with 409.03(b). A prime coat shall be applied after rolling and before overlay, see section 304-17.08.

Traffic will not be allowed on the cracking and seated pavement before the HMA base or intermediate courses are placed. The initial HMA course shall be placed within 48 hours of cracking and seating. In the event of rain prior to placing the overlay, the 48 hour time limitation shall be waived to allow sufficient time for the cracking and seated pavement to dry.

304-6.03(04) Unbonded PCCP Overlay over Old PCCP

Another effective way to rehabilitate a PCCP that has lost structural capacity is to overlay the existing PCCP with an unbonded PCCP. To create an unbonded PCCP a thin single layer, typically 1”, of HMA is placed on the existing PCCP. Prior to placing the QC/QA HMA, 5, 64, Intermediate, OG 9.5mm, a hydrated lime slurry for whitewashing shall be applied to the old PCCP. The use of Unbonded PCCP Overlay will require a unique special provision, in which case, the designer should contact the Pavement Engineering Manager.

Underdrains shall be designed and placed along the edges of the pavement prior to the overlay.

304-6.03(05) Full Depth Pavement Reclamation [Rev. Mar. 2018]

Full Depth Reclamation (FDR) is the a rehabilitation technique in which the full thickness of the asphalt pavement and a predetermined portion of the underlying materials (base, subbase and/or subgrade) is uniformly pulverized/reclaimed and blended to provide an upgraded, homogenous base material. The base materials are shaped, compacted, bladed, and prepared for the surface course. FDR depths vary depending on the thickness of the existing pavement structure, but generally range between 4 to 12 in. (100 to 300 mm). The INDOT Office of Geotechnical Services must investigate, evaluate, and make recommendation on the moisture content and Loss of Ignition
(LOI), to determine the suitability of the pavement subbase and subgrade materials for FDR. The INDOT Geotechnical Report data shall be used to develop a mix design with appropriate additives. FWD is also required to determine subbase and subgrade strength.

Pavements that have extensive subgrade or drainage problems are candidates for FDR only when additional work is undertaken to correct the deficiencies. In areas where the required treatment is too deep for single pass FDR or due to vertical constraints adjustments in the construction process can be made to address the constraints, such as a two-pass technique.

Reclamation of the existing asphalt bound pavement layers with the underlying materials produces a “granular” pavement layer which can be used as is, or additional stabilization can be achieved with the use of additional granular materials or chemical stabilizers.

Pavement distresses which can be treated by FDR include:

1. all forms of cracking including age, fatigue, edge, slippage, block, longitudinal, reflection, or discontinuity;
2. reduced ride quality due to swells, bumps, sags, patches, or depressions;
3. permanent deformations in the form of rutting, corrugations, or shoving;
4. loss of bonding between pavement layers;
5. moisture damage (stripping);
6. loss of surface integrity due to raveling, potholes or bleeding;
7. excessive shoulder drop off; or
8. inadequate structural capacity,

The expected design life, performance requirements during the design life, and acceptable future maintenance requirements are related to treatment depth of the FDR, types and amount of stabilizer used, subgrade type and conditions.

For FDR projects, an existing roadway assessment, structural capacity assessment, materials properties assessment, geometric assessment of the existing and proposed sections, traffic assessment, constructability assessment, and an economic assessment needs to be conducted.

The expected service lives of the various FDR rehabilitation techniques, when undertaking a life-cycle economic analysis, generally fall within the following ranges:

1. FDR with surface treatment ............. 7 years
2. FDR with HMA overlay ................. 7 - 15 years*

*MEPDG design analysis is necessary to determine the exact design life.
Additional information on the FDR process including project analysis, mix designs, construction requirements, and recommended specifications are included in the Basic Asphalt Recycling Manual (BARM) dated 2001, located on the Asphalt Recycling and Reclaiming Association (ARRA) webpage at: www.arra.org. The 2001 BARM recommended specifications, or Concrete Pavement Tech Center Guide, or other state’s specifications may be used to produce a Unique Special Provision in accordance with INDOT standard practice.

304-6.04 Preventive Maintenance

A Preventive Maintenance (PM) pavement treatment is intended to preserve and extend the service life of an existing good pavement. A PM project shall be considered as cost effective treatment to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without increasing structural capacity. The proper time for a PM is before the pavement experiences severe distress, structural problems, and moisture or aging-related damage. Projects that address deficiencies in the pavement structure or increase the structural capacity of the facility are not considered preventive maintenance. PM work includes surface treatments as described in Section 304-19.0, and mill and fill single layer HMA overlays.

304-6.04(01) Surface Treatment

Surface treatments are intended to preserve and extend the life of an existing good pavement. The pavement work on a surface treatment project extends the surface life, thus the functional life of the pavement. A surface treatment is intended to arrest light surface deterioration, retard progressive damage, improve friction, and reduce the need for routine maintenance. The proper time for a surface treatment is before the pavement experiences severe distress, structural problems, and moisture or aging-related damage. Surface treatments are as described in Section 304-19.0.

304-6.04(02) Mill and Fill Overlay Treatment

A mill and fill single layer HMA overlay treatment is to preserve and extend the life of an existing good pavement. INDOT’s typical mill and fill single layer HMA overlay consists of a surface course comprised of 1½” (165 lb/yd²) of a 9.5mm Mixture Designation, or a 2” thickness (220 lb/yd²) of a 12.5mm Mixture Designation. Prior to the overlay, the existing surface is removed using Milling, Asphalt to a required depth for the overlay. This treatment is used to replace a
deteriorated surface and retard progressive damage to lower layers in the pavement structure and reduce the need for routine maintenance.

304-6.04(03) In-Place Recycling

Reusing the existing materials and renewing the pavements through pavement recycling and reclaiming meets current social goals of providing safe and efficient roadways, while at the same time drastically reducing both the environmental impact and energy consumption specific to conventional pavement reconstruction. In-place recycling consists of two broad categories including Hot In-Place Recycling (HIR) or Cold Recycling (CR) which includes both Cold In-Place (CIR) or Cold Central Plant Recycling (CCPR).

Additional information on HIR and CR processes including project analysis, mix designs, construction requirements and recommended specifications are included in the Basic Asphalt Recycling Manual (BARM) dated 2001 located on the Asphalt Recycling and Reclaiming Association (ARRA) webpage at: www.arra.org. The 2001 BARM recommended specification or other State’s specifications may be used to produce a unique special provision in accordance with INDOT standard practice.

Hot In-Place Recycling

Hot In-Place Recycling (HIR) is the process of heating and softening the existing asphalt pavement for processing. HIR is limited in depth to less than 2 in. (50 mm) and will address oxidation (aging) and most surface related distresses, i.e., cracking confined to the surface of the pavement. After heating, the asphalt material is picked up and remixed with admixtures and then spread back onto the surface of the roadway and compacted, all in one operation. An HMA surface shall be placed after the HIR process. Pavements with structural distresses are not good candidates for HIR.

Cold Recycling

Cold recycling (CR) reuses the existing asphalt pavement by milling to a depth of 3 to 4 in. (75-100 mm), mixing the millings with a recycling agent (asphalt emulsion) and paving and compacting the cold-recycled mix. CR has been successfully used on pavements with a higher degree of cracking that would normally require removal of the cracked surface and a thick overlay. Instead, the top portion of the existing pavement is recycled and a thin overlay is applied over the cold recycled asphalt pavement. Cold Recycling, which includes both Cold In-Place Recycling (CIR) and Cold Central Plant Recycling (CCPR), is applicable for urban or rural roadways with
high or low volumes of traffic. The CIR process requires milling the existing pavement, mixing various recycling agents into the mixture, and then spreading the material across the pavement width for compacting. The CCPR process is the same except the material is transported to a Central Plant location for mixing and then is transported back to the site for placement and compaction.

CR can be used to rehabilitate pavement by addressing most types of pavement distresses. Cracked pavements which are structurally sound and have well-drained bases are the best candidates. The CR process destroys existing crack patterns and produces a crack-free layer for the new surface course such as an HMA or an asphalt surface treatment. For CR to be effective in mitigating cracking, as much of the existing asphalt pavement layer should be treated as possible.

For CR projects, an existing roadway assessment, structural capacity assessment, materials properties assessment, geometric assessment of the existing and proposed sections, traffic assessment, constructability assessment, and an economic assessment must be conducted.

### 304-7.0 PAVEMENT TYPE SELECTION

The pavement type for a project will be selected based on specific project considerations which include but are not limited to the project scope, the geotechnical engineering report, the project design traffic, LCCA, and the area of mainline pavement and shoulders that will be constructed.

1. If an LCCA between most effective HMA and PCCP alternatives shows that the present value of the more expensive of the options is more than 10% greater than that of the less expensive alternative, then the pavement type with the lower LCCA cost will be selected. That is the only pavement type to advance further on the project development/design process.

2. If the difference in LCCA present values of the HMA and PCCP alternatives is 10% or less, then an alternate bidding process will be used if the project is equal to or greater than 10,000 yd² of pavement area. There may be exceptions to this criterion if the Geotechnical Report recommends one type of pavement over the other due to in situ soil conditions or other considerations.

The Designer must submit the total square yards of the mainline and shoulder pavement area at least three weeks prior to letting date to Central Office, Senior Pavement Engineer. This is to provide enough time to determine the factor in the alternate bidding process prior to letting date.
3. An LPA or its representative can present an argument and justification to the Pavement Type Selection Panel for using one type of pavement over the other, HMA or PCCP, if requested. The Department will make the determination based on the argument and supporting documentation. The Pavement Type Selection Panel will be composed of the:

   a. Pavement Division Director;
   b. Capital Program Management Deputy Commissioner;
   c. Engineering and Asset Management Deputy Commissioner;
   d. Pavement Engineering Manager; and
   e. FHWA Pavement and Materials Engineer as a non-voting participant.

For LCCA unit costs of pavement pay items see the Pavement Engineering section on the Standards and Specifications webpage: http://www.in.gov/dot/div/contracts/standards/.

304-8.0 PAVEMENT TYPES

The types of pavement used in Indiana are aggregate, asphalt, Portland cement concrete, or composite (asphalt over concrete, or concrete over asphalt). A pavement designer should have a thorough understanding of these pavement types and their uses from pavement design courses, experience, and text books.

304-8.01 Aggregate Pavement

An aggregate pavement consists of a dense-graded compacted aggregate placed on a prepared subgrade. The pavement is typically composed of 4” compacted aggregate No. 73, on 6” compacted aggregate No. 53, on Subgrade Treatment, Type III or IIIA or as specified in Geotechnical Report, with appropriate drainage measures. Aggregate pavements are used on county roads, low-volume roads, and State Parks in Indiana. Aggregate pavements and bases located in Section 300 of the Standard Specifications.

304-8.02 Asphalt Pavement

A new asphalt pavement typically consists of a HMA surface course, on a HMA intermediate course, on either HMA base or a compacted aggregate base, directly on a prepared subgrade. An asphalt pavement overlay may consist of a surface course or a surface course on an intermediate course on the existing pavement. A drainage layer may be utilized near the bottom of a new asphalt
pavement if confined between two base layers or between an intermediate and base layer. Typical sections for HMA pavements are included in Section 304-21.

Lay or layer thicknesses are determined by the Nominal Maximum Aggregate Size, (NMAS) used in each mixture designation. Reference the table below entitled, Mixture Type and Maximum Particle Size. A layer thickness is what a pavement design engineer designs for a certain mixture designation. A layer may have to be divided into two or more lifts to accomplish proper construction and compaction. So a pavement designer must consider both the layer thickness and whether it needs to be divided into multiple lifts, and can these lifts be constructed and compacted accordingly.

Lay thicknesses play an important role in HMA construction quality control. Neither high lift thickness nor low lift thickness is desirable to achieve good compaction results. From a mechanistic point of view, the compaction pressure applied to the HMA layer is the highest at the top surface of the lift where the HMA materials directly contact the compacting roller. This compaction pressure decreases with depth, which means that if the lift thickness is too high, the required compaction pressure may not be applied to the materials at the bottom of the lift. On the other hand, since compaction is significantly affected by the lay down temperature and the temperature decreases more quickly with thin HMA lifts, good compaction result cannot be achieved either if the lift thickness is too low. In addition, there are many other factors that affect HMA compaction. Some of these factors are the nominal maximum aggregate size, aggregate gradation, and types of the asphalt binders. The Standard Specifications require that the finished thickness of any course shall be at least 2 times but not more than 4 times the maximum particle size as shown on the Design Mix Formula (DMF). This requirement applies during construction; however, the pavement designer should design the lay thickness according to the research findings from NCHRP – 531.

NCHRP - 531 indicated that the HMA pavement density that can be obtained under normal rolling conditions is clearly related to the ratio of thickness/NMAS of the HMA. To achieve proper compaction, the thickness/NMAS ratio should be 4, or the thickness/Maximum Particle Size should be 3. The pavement designer should target lifts of 3 times the Maximum Particle Size and avoid designing to the minimum or maximum. Likewise the pavement design should specify the smaller aggregate size for intermediate and base mixtures where given the choice. While this will require more binder, this makes for a more desirable pavement structure: better density, better stability, and less permeability. If the design layer thickness of a specific layer exceeds 4 times the Maximum Particle Size, then that specific layer should be laid in two lifts, e.g., 770 lb./yd² of Base, 19.0 mm shall be laid in two lifts of 385 lb./yd² each lift.
MIXTURE TYPE AND MAXIMUM PARTICLE SIZE

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Nominal Maximum Aggregate Size (NMAS)</th>
<th>Maximum Particle Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5 mm</td>
<td>0.375” (3/8”)</td>
<td>0.5”</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>0.5”</td>
<td>0.75”</td>
</tr>
<tr>
<td>19.0 mm</td>
<td>0.75”</td>
<td>1.0”</td>
</tr>
<tr>
<td>25.0 mm</td>
<td>1.0”</td>
<td>1.5”</td>
</tr>
</tbody>
</table>

Thickness, maximum particle size, and PG Binder are determined based on various factors of the roadway being evaluated. The Pavement Designer may choose various maximum particle size and appropriate lay rates for Surface, Intermediate, and Base Courses based on design criteria and in accordance with Section 400 of the *Standard Specifications*. The PG Binder grade is determined using LTPPBind software utilizing data from National Weather Service (NWS) weather stations.

Revisions included in the 2014 *Standard Specifications* require HMA pavement to incorporate the following:

1. safety edge on Surface and Intermediate layers that are constructed adjacent to an aggregate or earth shoulder;

2. longitudinal joint adhesive on Surface and Intermediate layers;

3. liquid asphalt sealant, AE-F, 24 in. wide and applied on the Surface layer over the longitudinal joint; and

4. base seal under all Open-Graded HMA. See INDOT *Standard Specifications*, Section 415.

Centerline Rumble Strips should be considered on all two-lane roads.

HMA pavement surfaces comprise the majority of pavements seen on Indiana’s highway system. They are used on local roads, state routes, on the NHS, and interstate highways. INDOT’s typical full-depth asphalt pavements are composed of a surface, on intermediate, on base, on OG base, on base, on prepared subgrade, with underdrains with adequate support from foundation soils below.
304-8.02(01) HMA / SMA Surface

INDOT’s typical surface course is comprised of 1½” (165 lb/yd²) of a 9.5 mm mixture designation. A thicker 2” (220 lb./yd²) course of a 12.5 mm mixture designation may also be used when required. These typical surface courses may be used as PM treatments, in a two-lift minor structural pavement treatment, or in a full-depth pavement. INDOT also uses a thin-layer HMA surface, comprised of a ¾” thickness of a 4.75mm mixture designation, as a PM surface treatment. The PG binder grades for the surface course can be PG 64-22, PG 70-22, or PG 76-22. These surfaces are used on all functional classifications of roads in Indiana.

304-8.02(02) HMA Intermediate

INDOT’s typical intermediate course is comprised of 2½” (275 lb/yd², but may be as much as 4” at 440 lb/yd²) composed of a 9.5 mm or 12.5 mm mixture designation, although a 19.0 mm or 25.0 mm mixture may also be used, with PG 64-22, PG 70-22, or PG 76-22. The pavement designer must always be cognizant of the lay rate/maximum aggregate (particle) size relationship and target a lay rate of 3 times the maximum particle size. These typical intermediate courses may be used in a two-lift pavement treatment or in a full-depth pavement. These intermediate courses are used in all functional classifications of roads in Indiana.

304-8.02(03) HMA Base

INDOT’s typical base course is comprised of thicknesses ranging from 3” to 6”, (330 lb/yd² to 660 lb/yd²) composed of a 19.0 mm or 25.0 mm mixture designation with PG 64-22. These typical base courses are to be used in a full-depth pavement. These base courses are used in all functional classifications of roads in Indiana.

304-8.02(04) HMA Open Graded Drainage Layer

If underdrains are warranted, then an open graded (OG) drainage layer is most likely required. QC/QA-HMA 5, 76, Intermediate, OG 19.0 mm is used as a conduit to remove water entering the pavement system. INDOT’s typical lay rate for OG layer is 100 lb/yd² per inch and is typically placed at 250 lb/yd² (2½”). This is also a structural layer to help carry the anticipated design traffic loads. If open-graded mixtures are specified, a dense graded base mixture shall be specified under the open-graded layer, and underdrains must be included. Prior to placing an OG mixture, the underlying HMA course shall have a full width base seal applied in accordance with INDOT Standard Specifications, Section 415.
304-8.02(05) Compacted Aggregate Base

HMA over compacted aggregate pavement will be designed as flexible pavement. See Figures 304-21F, 304-21G, 304-21M, and 304-21N for specific details.

The project designer should use the appropriate mixture designations shown for QC/QA-HMA or HMA mixtures in accordance with Section 304-15.0. The compacted aggregate should be as designed within the limits shown in the 304-21 series of figures.

The compacted aggregate base functions as a structural layer while economically increasing the pavement thickness to help protect the pavement from the effects of frost action. Compacted aggregate bases are used under aggregate, HMA, or PCC pavements. See INDOT Standard Specifications, Section 301, including any recurring special provisions for aggregate bases. A compacted aggregate is typically used on shoulders; see INDOT Standard Specifications, Section 303, for aggregate pavements or shoulders.

304-8.03 Portland Cement Concrete Pavement

Portland cement concrete pavement (PCCP) consists of concrete materials on Subbase for PCCP, or on Dense Graded Subbase, on a treated subgrade. PCCP is composed of portland cement, pozzolanic materials (such as fly ash), coarse and fine aggregates, water, and chemical admixtures. Dowels are constructed at transverse planned joints to provide load transfer between adjacent panels, and tie bars are placed along longitudinal joints to provide lateral support, tying two lanes together. Safety edge shall be constructed where the pavement is constructed adjacent to earth or aggregate shoulder.

PCCP is typically used on the Interstate system and the NHS, particularly where there are high volumes of traffic, especially trucks. PCCP is also used on state routes and in urban areas. INDOT has exclusively constructed Jointed Plain Concrete Pavements (JPCP) in the last three decades. Continuously Reinforced Concrete (CRC) Pavements were initially constructed on the Interstate system in Indiana in the 1960’s and1970’s. However, CRC Pavements in Indiana were constructed without drainable subbases and without underdrain systems and had inherent subgrade problems and began pumping and consequently “punch-outs” occurred. There are very few bare CRC Pavements remaining; most CRC pavements have been covered with HMA or PCCP. Use of CRC pavement can still be considered if traffic and economic considerations show it is the best alternative for a project. See Section 304-16.02 for CRC. See INDOT Standard Specifications, Section 500, including any recurring special provisions, for concrete pavement.
Subbase for PCCP consists of two layers of aggregate placed under PCCP to prevent pumping of erodible subgrade material and to provide support for the pavement. The two layers are composed of a 3” OG aggregate, No. 8 on 6” dense graded compacted aggregate No. 53. A drainable subbase provides a conduit to remove water that enters the pavement system and should be used for pavement where underdrains are required. Dense Graded Subbase is used under PCCP where underdrains are not used. A dense graded subbase provides for a stable working platform together with support for the pavement without drainage layers. See INDOT Standard Specifications, Section 302, including any recurring special provisions, for subbase.

304-8.04 Composite Pavement

A composite pavement consists of multiple pavement types, i.e., HMA over PCCP or PCCP over asphalt. A composite pavement should be designed in accordance with MEPDG using AASHTOWare Pavement ME software. The majority of INDOT pavements today are composite pavements. Special attention should be used when patching, widening, overlaying, or otherwise rehabilitating composite pavements. A pavement designer should match the existing pavement composition, if possible when patching and widening composite pavements. Additional testing is usually required to determine the strength parameters of a composite pavement. Cores are always required to define the composition of a composite pavement.

304-9.0 PAVEMENT DISTRESSES

The strengths and limitations of each pavement system must be understood prior to designing a pavement. The type, extent, and severity of pavement distresses and their causes and recommended treatments should be well known. See Distress Identification Guide, LTPP, FHWA Publication Number: FHWA-RD-03-031, latest version, for additional information.

Types of distresses related to aggregate pavement are as follows:

1. Dusting
2. Potholing
3. Rutting
4. Washboarding

Types of distresses related to asphalt pavement are as follows:

1. Block Cracking
2. Bleeding
3. Blowup – On Composite Pavement with Concrete below HMA
4. Edge Cracking
5. Fatigue Cracking
6. Frost Heave
7. Longitudinal Cracking
8. Longitudinal Joints Open
9. Potholes
10. Polishing
11. Raveling
12. Reflective Cracking
13. Rutting
14. Shoulder Drop-off
15. Shoving
16. Stripping
17. Thermal Cracking
18. Transverse Cracking – Top-Down or Bottom-Up
19. Weathering.

Types of distresses associated with concrete pavement are as follows:

1. Alkali-Silica Reactivity (ASR)
2. Blowup
3. Corner Break
4. Durability Cracking ("D" Cracking)
5. Faulting
6. Joint Failure (including Longitudinal Joint related to De-Icing Chemicals)
7. Longitudinal Cracking
8. PCCP Joint-Seal Failure
9. Polishing
10. Poor Rideability
11. Pop-out
12. Pumping
13. Punch-out
14. Transverse Cracking
15. Scaling
16. Spalling
17. Structural Failure
304-10.0  PAVEMENT MILLING

An asphalt or concrete pavement may be milled to remove distressed layers of material, make crown corrections, maintain curb height or vertical clearance, scarify existing surface, surface profiling, removal of asphalt overlay, or to provide a pavement transition. See INDOT *Standard Specifications*, Section 306, including any recurring special provisions, for milling. The types of pavement milling and their applications are as follows:

1. **Asphalt or PCCP Scarification Milling.** Scarification milling is used to roughen the surface or remove excessive crack sealant prior to placing an HMA overlay.

2. **Asphalt or PCCP Profile Milling.** Profile milling is used to correct a cross-slope (crown) deficiency.

3. **Approach Milling.** Approach milling is used to provide a smooth connection between an overlay and driveways, commercial or public-road approach, and mailbox approaches.

4. **Asphalt or PCCP Milling.** Asphalt or PCCP milling is used to remove distresses near the surface of the pavement or prior to placing an HMA inlay.

5. **Asphalt Overlay Removal.** Asphalt overlay removal is used to remove asphalt materials down to a concrete or brick base.

6. **Transition Milling.** Transition milling is used to provide a transition to an adjoining section.

304-10.01  Asphalt or PCCP Scarification Milling

Asphalt or PCCP scarification milling is used to provide a roughened texture to an existing surface. Asphalt or PCCP scarification milling will remove crack sealant to prevent slippage of the overlay materials or roughen the existing surface that has polished due to traffic. Milling operations to correct pavement conditions that require deeper milling should be in accordance with Section 304-10.04.

Asphalt or PCCP scarification milling is generally used to prepare an existing pavement for a single-course HMA overlay. Asphalt or PCCP scarification milling is used to prepare an existing pavement for a functional overlay if the existing pavement has excessive crack sealant or requires minor profile corrections.
304-10.02 Asphalt or PCCP Profile Milling

Asphalt or PCCP profile milling is used to correct minor profile or cross-slope (crown) deficiencies.

304-10.03 Approach Milling

The application of approach milling is used to provide a connection between an overlay and driveways, commercial or public-road approach, and mailbox approaches. The transition slope and notch depth in the existing asphalt or concrete pavement will be in accordance with the INDOT Standard Drawings.

Approach milling is not to be performed at driveways unless it is required to meet a paved surface that continues beyond the construction limit. If the driveway is other than HMA or PCC beyond the construction limits, the approach milling is not required.

304-10.04 Asphalt or PCCP Milling

Asphalt or PCCP milling is intended to remove material from an existing pavement to a specified average depth by milling the surface and creating a uniform profile. An average depth of milling should be specified depending on the condition of the pavement or project requirements. Asphalt and PCCP milling maybe used in the following cases:

1. prior to placing an HMA or PCCP inlay;
2. to correct substandard cross slope or crown condition;
3. profile correction; or
4. to maintain vertical clearance or curb height.

In addition to the cases listed above, Asphalt milling may be used for the removal of stripped or distressed asphalt.

The average milling depth specified will be sufficient to accommodate the HMA inlay, or the removal of distressed materials, and to achieve the desired cross slope. For a variable milling depth to correct a cross-slope deficiency, the limits and associated milling depths must be shown on the typical cross sections in accordance with the series of Figures 304-21.
304-10.05 Asphalt Overlay Removal

Asphalt overlay removal consists of milling to remove an entire asphalt overlay from a concrete or brick base. The designer will designate the approximate existing asphalt thickness on the typical cross sections. The designer should be aware that milling can dislodge or loosen bricks and result in construction challenges. To avoid construction issues associated with bricks, it is recommended to allow a sufficient amount, at least 2 in. or more, of existing asphalt pavement to remain in-place to keep the bricks stable.

304-10.06 Transition Milling

Transition milling is used to provide a connection between an HMA overlay and an adjoining pavement, paving exception, or at the beginning and end of the paving project. The transition slope and notch depth in the existing asphalt or concrete pavement will be in accordance with the INDOT Standard Drawings.

304-11.0 Pavement Patching

The project manager and the pavement designer to determine and overcome critical project challenges such as maintenance of traffic (MOT), pavement patching, temporary pavements, drainage (underdrains), etc. The pavement designer must be aware of the requirements and follow the Lane Closure Policy of the Department, or request an exception with adequate justification. The Pavement Designer is responsible for specifying the composition, depth, and location of various patch types.

The Pavement Designer shall produce a Patching Table to assist in the proper design and construction of the patches on the project. A Patching Table including start station locations and end station locations of the patches, lane (travel, passing, mainline, shoulder, approach, etc.), direction (NB, SB, etc.), length (ft), width (ft), and area (yd²) shall be shown on the Plans. Separate tables shall be produced for partial depth patches and full depth patches.

See Typical Patch Sections Figures 304-21CC, 304-21DD, 304-21EE, and 304-21FF.
The Pavement Designer must coordinate with the roadway designer to determine MOT with consideration of how long a lane can be closed during the patching operation. See Chapter 82 for work zone traffic control considerations.

All PCCP patches shall be doweled to the existing remaining concrete pavement. A 6-ft minimum and 15-ft maximum spacing shall be between two D-1 joints. If a 20-ft panel is replaced, a D-1 joint shall be installed at 10 ft. Whenever possible, match the transverse joints with adjacent lanes. Patches less than one panel in length do not need to be tied to the existing concrete pavement. For a 20-ft original concrete joint spacing, the slab movement will be about 1/12 ft. So for every 60 ft of patching length (intermittent or continuous patching), ¼-in foam should be placed.

PCCP Patching using 502 mixture can be opened to traffic as follows:

1. Construction vehicles and equipment after 10 days or when test beams indicate a modulus of rupture (flexural strength) of at least 550 psi.

2. Non-construction traffic after 14 days or when test beams indicate a modulus of rupture (flexural strength) of at least 550 psi. All cracks and joints must be sealed.

The 550 psi strength can typically be reached in 2 days (48 hours), when using 502.04(b) High Early Strength (HES) concrete. This allows for a 12-hour curing time. It is recommended to use 502 HES concrete on long patches (> 16’).

The designer should only use 506 PCCP Patching mix when a lane must be open during the daylight hours and the contractor only has from 6:00 p.m. to 6:00 a.m. for night construction. 506 mixes should only be used on maximum 16-ft long patches. See INDOT Standard Specifications, 506.11 Opening to Traffic.

The integrity of the underdrains should be maintained in the patching process. The designer should incorporate the underdrain details in the design; including sequencing of the patching process in conjunction with underdrain installation. Patching should not disturb or block/clog underdrain.

A minimum of 6 years of service life should be expected for a patching project. For a PCCP structural pavement treatment project if the patching is over 8%, an LCCA should be completed to compare a PCCP overlay or slab-reduction technique, e.g., PCCP rubblization with an HMA overlay. Also if patching is at or above 30%, a reconstruction project should be considered as an alternate treatment based on economic analysis.

Use inverted-T patches where dowels cannot be drilled into the old slab. Dowels cannot be secured into an old PCCP concrete slab if it has been cracked and seated, rubblized, or cores indicate it has significant deterioration. See Figures 304-21EE and 304-21FF for typical sections.
304-11.02 HMA Patching

HMA partial-depth patch may be used where the deterioration is only in the upper one or two layers of existing HMA. A partial depth patch has been historically referred to as “drop the drum”, since the removal process is accomplished by the milling machine dropping the milling drum deeper in the location of the patch, typically to a 3” to 6” depth. Then the hole is filled with new HMA (HMA Patching, Type__), as this process is usually included in an overlay project. After the final milling is accomplished the HMA surface is applied.

Longitudinal patching is usually required in a widened area, where the existing asphalt surface has an open longitudinal crack or joint at the mainline/widened interface. A longitudinal patch should be placed by milling or otherwise removing the upper one or two layers of HMA and installing new HMA. Depth and width of the patch is critical to assuring that the new HMA can be installed with proper density. Reference INDOT Standard Specifications, Section 304.

Patch width is critical in order to obtain proper compaction. A vibratory roller is typically 8’ wide. There are smaller non-vibratory rollers as small as 4’ wide, and even smaller hand-controlled “jumping jack” compactors. Only the vibratory roller can definitively compact the HMA to the desired density. The Pavement Designer should avoid designing small patches having a width less than 2’.

The integrity of the underdrains should be maintained in the patching process. The designer should incorporate the underdrain details in the design; including sequencing of the patching process in conjunction with underdrain installation. Patching should not disturb or block/clog underdrain. The pay item is “HMA Patching, Type____,” per INDOT Standard Specifications, Section 304.

304-11.03 Composite Patching

Composite patches should always match existing pavement composition and depths where practical. The Pavement Designer will determine the most appropriate patch composition and depth. Patch width is very critical for achieving the proper compaction. The integrity of the underdrains should be maintained in the patching process. The designer should incorporate the underdrain details in the design; including sequencing of the patching process in conjunction with underdrain installation. Patching should not disturb or block/clog underdrain. See Figure 304-21EE for typical section.

304-12.0 PAVEMENT WIDENING
Where pavements are being widened in an overlay project the widening is brought up to the elevation of the existing pavement and an overlay is constructed over the widened area and the existing pavement simultaneously for continuity. The depth of the widened area must be at least that of the adjacent pavement so that the subgrade is not stepped. In excessively thick sections compacted aggregate may be substituted for a portion of the HMA. Prior to overlay the area of the existing pavement and new widening are scarification/profile milled to make a smooth plane for the overlay.

All costs for widening pavements 5 ft or less are included in INDOT *Standard Specifications* Section 304 Widening with HMA or Section 305 Widening with PCCP Base. All costs for widening pavements 5 ft or more are determined based on the individual components of the work.

**304-12.01 Widening with HMA**

An existing pavement may be widened to 5 ft or less on each side if widening with HMA. However, the minimum width of widening with HMA specified is 2 ft for constructability purposes. This minimum width of widening may result in extra lane width or may require removal of existing pavement to satisfy the 2-ft minimum-width requirement. The longitudinal joint of the widened pavement should not be located in the wheel path of the driving lane. The pay-item designation for this work will be in accordance with Section 304 of INDOT *Standard Specifications*, Widening with HMA, of the type required, regardless of the quantity involved.

If specific project widening requirements exceed 5 ft, the widened pavement area will not be specified as HMA widening, but will be identified as HMA pavement. The pay items specified will be QC/QA-HMA in accordance with Section 401, and the excavation and subgrade treatment will be as described in the INDOT *Standard Specifications*, Section 207. If the existing pavement has underdrains and OG layer, the widening design shall perpetuate the underdrain system.

**304-12.02 Widening with PCC Base**

If widening of the pavement is needed and the existing subbase is open graded, the widened PCC base will utilize Subbase for PCCP. If the existing subbase is dense graded, the widened PCC base will utilize Dense Graded Subbase. The width of PCC base widening is limited to pavement widening of less than or equal to 5 ft. An existing pavement may be widened up to 5 ft on both sides with PCC base. The pay item designation of this work will be Widening with PCC Base, ___ in., in accordance with the INDOT *Standard Specifications*, Section 305.
304-12.03 Widening for Composite Pavements

Widening of asphalt over PCC (composite) pavement will be designed to match the existing pavement. If widening of the pavement is needed and the existing subbase is open graded, the widened PCC base will utilize Subbase for PCCP. If the existing subbase is dense graded, the widened PCC base will utilize Dense Graded Subbase. See INDOT Standard Specifications, Section 302.

It may not be cost effective or practical to widen a composite pavement by 2’ to 5’ for one or two miles. Also the old concrete may not be in suitable condition to install tie bars. There must be good engineering analysis to determine the cost effectiveness or practicality of the widening. In situations such as this, widening with HMA should be considered.

304-13.0 PAVEMENT TESTING

INDOT does pavement testing to determine strength, cause of failures, and basic forensics. These tests include, but are not limited to:

1. Falling Weight Deflectometer (FWD)
2. Ground Penetrating Radar (GPR)
3. Coring
4. Friction

304-13.01 Falling-Weight-Deflectometer (FWD) Testing

The pavement designer should evaluate the need for FWD testing pertaining to concrete, asphalt, or composite pavement. The FWD data is used, but not limited to, evaluating the structural adequacy of an existing pavement section, evaluating pavement shoulder adequacy for temporary traffic, or providing an estimated quantity of underseal to be included in the plans for an existing PCCP over dense graded subbase. The INDOT Deflection Testing Request Form is available on the Department’s website at [www.indotresearch.org](http://www.indotresearch.org).

FWD testing must be requested at no later than the 30% stage of plan development (Stage 1). FWD testing cannot be performed during the winter months, typically between late October and late April. This must be taken into account when requesting the testing. The district should coordinate traffic-control activities for the FWD testing.
304-13.02 Ground Penetrating Radar (GPR) Testing

The pavement designer should evaluate the need for GPR testing pertaining to concrete, asphalt, or composite pavement. The GPR data is used to detect anomalies and moisture under the pavement structure and also to detect the drainage system and reinforcing steel locations. GPR works best where moisture is present. The INDOT GPR Request Form is available on the Department’s website at www.indotresearch.org.

304-13.03 Pavement Coring

The pavement designer should evaluate the need for pavement coring. Cores are required for all pavement rehabilitation projects and the information should be requested in advance of the date it is required for project selection. Cores are used to verify the thickness, composition, structural condition, and forensic evaluation of the existing pavement. Forensic cores may need to be tested by INDOT Materials Management for material quality, and these tests are to be arranged by the district or Central Office Pavement Engineer. Cores for history or thickness verification should be taken in locations of sound pavement. Cores must be obtained during non-freezing temperature periods due to equipment and safety concerns. The District should coordinate traffic-control activities for the coring, as much as possible. For an LPA project, the LPA shall be responsible for coring and traffic control. A Core Report should be produced that discusses the findings of the cores and core photographs should be included.

Cores should be located as follows:

1. at least every ½ mile in each lane;
2. at select joints with distresses;
3. in existing widened areas where it is obvious there is a different pavement structure than the mainline;
4. at cracks to determine top-down or bottom-up cracking; and
5. at predominant distress locations.
304-13.04 Pavement Friction

Wet weather pavement friction is one of the most important components providing safety on the nation’s highways. Pavement friction is provided through two primary characteristics of the pavement surface; microtexture and macrotexture.

“The microtexture is the fine scale texture of the aggregate particles themselves, usually defined as less than 0.5 mm. Microtexture determines the friction of the pavement surface at low speeds. Macrotexture is large scale texture, on the order of 0.5 to 50 mm, provided between the aggregate particles. Macrotexture provides channels through which surface water can flow, providing surface drainage and improving the contact between vehicle tires and the pavement. This texture helps to decrease the chances for a vehicle to hydroplane in wet weather. In general, as the speed of the vehicle increases, the friction decreases, but the rate at which it decreases depends on the pavement macrotexture and how quickly water can be forced out of the tire-pavement interface. Higher macrotexture allows for more rapid drainage of the water and therefore a higher friction value.”

(McDaniel, R. S., Investigating the Feasibility of Integrating Pavement Friction and Texture Depth Data in Modeling for INDOT PMS, JTRP SPR-2936, October 2012, p.2.)

The INDOT Research Division annually conducts approximately 6,700 lane-miles of friction testing using ASTM E274 Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire, (towed friction trailer). The towed friction trailer uses smooth tires and applies water just in front of the tires as the test progresses at a constant speed, 30, 40, and 50 mph. When the friction number (FN) is below a so-called friction flag value, the District is informed and the site is visited to determine if remediation is needed. Friction testing may be requested of the INDOT Research Division if the district Pavement Engineer or district Maintenance personnel deem it necessary to help evaluate the pavement for proper treatment. See www.indotresearch.org for more information.

304-14.0 MECHANISTIC EMPIRICAL PAVEMENT DESIGN GUIDE

The Mechanistic Emperical Pavement Design Guide (MEPDG), AASHTO's pavement design guide, shall be used for the design of each pavement structure. The design process is based on the predictive performance of a pavement section to be designed to predefined parameters identified as failing. The pavement design itself is an iterative process where the pavement designer selects a cross section for the pavement based on economic benefits, performance, maintenance, and constructability. The objective of pavement design using the MEPDG process is to make iterative
inputs to the input parameters identified as important and critical, process the inputs to determine the pavement performance prediction, and compare the pavement-performance prediction with a preset performance requirement. Therefore, more than one design may satisfy the preset performance requirement. The final design must satisfy the performance-indicator criteria (threshold value) and design reliability level for the project and be economically justified.

This section discusses the typical factors and inputs to be used with the MEPDG using AASHTOWare Pavement ME Design software formerly known as DARWinME. The organization of the factors and inputs is based on the various sections of the software.

### 304-14.01 MEPDG General Inputs Using AASHTOWare Pavement ME Design software

1. **General Information.**
   
   a. **Design Type.** Select from one of three options; New Pavement, Overlay and Restoration. If performing alternative analyses, save the project with a different name before changing it to a different design type.
   
   b. **Pavement Type.** When analyzing a new pavement, choices are Flexible Pavement, Jointed Plain Concrete Pavement (JPCP), and Continuously Reinforced Concrete Pavement (CRCP). When analyzing an overlay, choices are Asphalt Concrete (AC) over AC, AC over JPCP, AC over CRCP, various bonded concrete overlays, and unbounded concrete overlays over AC, JPCP and CRCP. The Restoration design type is for JPCP restoration analysis. The software is only capable of analyzing for one existing pavement type, so when looking at an overlay of an existing HMA over concrete pavement, there are two ways to set up the analysis as either AC over AC with concrete being seen as crushed stone with the resilient modulus shown from the FWD results, or AC over the concrete pavement representing the existing AC as new AC with the properties of the existing pavement.
   
   c. **Design Life, Years.** See Figure 304-14A, Pavement Design Life, for pavement performance periods. The design life can also be set to a higher value that insures failure to determine the actual mode and time of failure.
   
   d. **Base Construction/Existing Construction.** This is the month and year of the scheduled construction of the granular base or prepared subgrade for new pavements or the existing pavement construction year for overlays or restorations. The base construction input is only for a new HMA pavement and is an important
parameter used to calculate the predictive performance of the pavement. For an INDOT project, it is to be assumed that the base or subgrade will be constructed in May. The existing construction month and year would be based upon the initial construction date for concrete pavements or the date for the construction of the layer that is at the bottom of the proposed mill for an HMA pavement.

e. **Pavement Construction Month.** This is the month and year of the scheduled placement of the HMA or PCCP. This input is an important parameter used to calculate the predictive performance of the pavement. For an INDOT project, it is to be assumed that the pavement will be constructed in July.

f. **Traffic Open Month:** This is the month and year of the scheduled opening to traffic upon completion of the project. This input is an important parameter used to calculate the predictive performance of the pavement. For an INDOT project, it is to be assumed that the pavement will be opened to traffic in September. Traffic Open Month must be more than one month after Pavement Construction Month.

2. **Analysis Parameter.** The inputs in this section are important and sensitive to the analysis and the final design of the project.

a. **Initial IRI (in./mi).** This is the predictive International Roughness Index for newly-constructed pavement. A typical value is 70 in./mi for both HMA and PCCP surfaces.

b. **Performance Criteria for Pavement Design.** These are the performance criteria used for acceptability of the iterative trials. These are the most important inputs that the trial design must achieve or exceed. Performance criteria for asphalt pavement is shown in Figure 304-14B. Performance criteria for concrete pavement is shown in Figure 304-14C. The failure modes of top-down cracking and total deformation are not to be considered for INDOT pavement design and should be ignored in the analysis.

3. **Traffic Inputs.** Determine Traffic Group A, B, C, or D from Item 4.a. below. Prior to importing the AADTT and other inputs such as the number of lanes in the design direction, import the site-specific traffic group data from the traffic input files. See Item 4 for more information related to traffic. Check to make sure the value from the import is correct for the specific project.
4. **INDOT Default Traffic-Distribution Input Files.** These are available to an INDOT designer within the Citrix drive location for the software through a shortcut under the user name of each designer. The information is also available to a pavement designer outside of INDOT on the Department’s website. The user of the software will be able to import the Traffic Volume Adjustment, Axle Load Distribution Factor, and the General Traffic Inputs.

   a. **Initial Two-Way AADTT.** This value is the Average Annual Daily Truck Traffic (AADTT) after the roadway is opened to traffic or the rehabilitation has been completed. It represents both directions and all lanes. For ramps and one way roads double the AADTT input. Traffic is divided into four Truck Weight Road Groups (TWRG), based on the AADTT, as follows:

   1) Traffic Group A, AADTT ≤ 3,000
   2) Traffic Group B, 3,000 < AADTT ≤ 6,000
   3) Traffic Group C, 6,000 < AADTT ≤ 20,000 AADTT
   4) Traffic Group D, AADTT > 20,000

   The TWRG traffic import should be made before any actual traffic information is entered because it will be overwritten at the time of the import.

   b. **Number of Lanes in Design Direction.** This is the number of traveled main lanes in the design direction, not including the acceleration, deceleration, or turn lanes.

   Example: 4-lane road with 2 lanes east bound and 2 lanes west bound. The input is 2. Check to make sure the value from the import is correct for the specific project.

   c. **Percent of Trucks in the Design Direction.** This value represents the percentage of trucks in the design direction relative to all trucks using the roadway in both directions. The recommended values are as follows:

   1) 2-Lane Road, 52%
   2) 4-Lane Road, 55%
   3) 6-Lane Road, 55%
   4) 8-Lane Road, 57%
   5) 10-Lane Road, 55%
   6) 12-Lane Road, 54%

   Check to make sure the value from the import is correct for the specific project.
d. **Percent of Trucks in the Design Lane.** This value represents the percent of trucks of FHWA Class 4 and above in the design lane in the design direction relative to all trucks using the roadway in both directions.

1) 2-Lane Road, 100%
2) 4-Lane Road, 90%
3) 6-Lane Road, 60%
4) 8-Lane Road, 45%
5) 10-Lane Road, 40%
6) 12-Lane Road, 40%
7) 1-Lane Ramp or Street, 100%
8) Multi-Lane Ramp, 90%
9) Multi-Lane One-Way Street, 90%

Check to make sure the value from the import is correct for the specific project.

e. **Operational Speed.** This value represents the posted truck speed limit. The posted speed limit should be used for all traffic groups.

f. **Growth Rate.** This value represents the growth of truck traffic during the pavement life. Assuming that the growth rate is uniform over time, the rate of increase remains the same throughout the design period.

The recommended growth method is linear growth. The project-specific Traffic Growth Factor should be used. If the growth rate from the import does not match the given growth rate for the project, the corrected growth rate must be entered for each class of truck.

g. **Design Lane Width.** This value represents the width of the through lane. The input entry is in the Lateral Wander section of the traffic inputs. The default lane width is set to 12 ft. If the lane width for a specific project is less than 12 ft, enter the correct width. If the lane width is more than 12 ft, e.g., on a single lane ramp, then leave this entry as 12 ft.

h. **Mean Wheel Location.** This value represents the distance for the outer edge of the wheel to the edge travel lane. The change in design lane width requires changes to the mean wheel location. If the design lane width is 10 ft then the mean wheel location would be 6 in. If the design lane width is 11 ft, then the mean wheel
location would be 12 in. The default value for the mean wheel location in a 12-ft lane is 18 in.

i. **Axle Distributions.** Single, tandem, tridem and quad-axle distributions are available for each TRWG and must be imported.

5. **Climate.** The climate inputs in the AASHTOWare Pavement ME Design software are based on the project location. A climatic data file must be generated for each pavement design project. There are 7 specific climate inputs from 7 weather stations in Indiana, and most are in major cities. There are also weather stations in surrounding states that might be closer to a project. Virtual weather stations can be created. When the latitude and longitude of the project are entered, the software will present the closest weather stations. If a weather station has missing data, this can be corrected. Specific climate data has to be imported from a file.

6. **Depth of Water Table.** Use the water table depth as shown in the soil boring logs, appropriately adjusted for soil types whose capillary action may raise the water table and climactic conditions. An example of climactic conditions would be if soil boring was taken during a drought which would not accurately represent the typical water table.

### 304-14.02 Flexible Pavement Layer Design

A flexible pavement system consists of HMA, i.e., Surface, Intermediate, or Base, on an unstabilized base layer or prepared subgrade. A final pavement structure design in the AASHTOWare Pavement ME Design software should include all of the layers shown in the series of Figures 304-21 typical pavement sections. Iterations should be conducted to minimize the HMA thickness while satisfying the pavement performance prediction for the design life. Optimal design and then one failure iteration must be submitted for review for each design.

Each flexible pavement structure design using the software should be performed using the specific imports for the District that the project is within, the PG grade and the mixture type being used on each layer which is available to an INDOT designer within the Citrix drive location for the software with a shortcut to it under the user name of each designer. The information is also available to a pavement designer outside of INDOT on the Department’s website.

1. **Structure.**

   a. **Surface Shortwave Absorptivity.** Use the software default value of 0.85.
b. Layers. The layer inputs should be based on the INDOT *Standard Specifications* and Figures 304-21B, 304-21E, 304-21F, and 304-21G. A maximum of 6 input layers is supported. The maximum number of input layers that can be asphalt layers is 3. If the trial design includes a drainage layer, the bottom Base layer and drainage layer should be input as one dense graded Base layer, or if there is a Base layer on top and bottom of drainage layer, combine all three as one dense graded Base layer. The layer thicknesses should be in accordance with Figures 304-21A, 304-21C, 304-21D, and 304-21H.

1) Type, Material, and Thickness. These are determined based on the INDOT *Standard Specifications* and Figures 304-21B, 304-21E, 304-21F, and 304-21G for the trial design.

2) In HMA pavement with OG layer, the bottom most HMA layer (between OG and subgrade treatment) must be 3 in. thick.

3) The thickness of any HMA layer should not be at the minimum or maximum lay rate for that material. The designer should target the total thickness of each layer in increments of 2 to 4 times the maximum particle size, preferably 3 times (4 times the nominal maximum aggregate size).

4) If the pavement design differs from typical sections shown in this chapter, approval is required from the Pavement Engineering Manager. Include a sketch or modified drawing with the pavement design and justification for the change.

5) Bedrock. This can be ignored where the depth to it is greater than 20 ft.

6) Filter Fabrics. INDOT does not consider filter fabric to be a drainage layer. For drainage purposes in the pavement structure, these should be ignored in the software design.

2. Material Properties. The Division of Pavement will maintain input files associated with material properties. The input files will be available to an INDOT designer within the Citrix drive location for software with a shortcut to it under the user name of each designer. The information is also available to a pavement designer outside of INDOT on the Department’s website. The user of the software will be able to import the following from each file:
a. dynamic modulus;

b. SuperPave Asphalt Binder Test data; and

c. asphalt general inputs. For 25.0 mm, 19.0 mm, 12.5 mm, 9.5 mm NMAS, or SMA 9.5 mm NMAS, values are provided in Figure 304-14D.

3. Thermal Cracking. For aggregate coefficient of thermal contraction, the range is from $4 \times 10^{-6}$ to $8 \times 10^{-6}$. Typically the value for limestone is $6.05 \times 10^{-6}$.

**304-14.03 Rigid Pavement Layer Design**

A rigid pavement system consists of portland cement concrete pavement on granular subbase on prepared subgrade. A final pavement structure design using the AASHTOWare Pavement ME Design software should include all the layers described in the typical sections shown in the series of Figures 304-21. Iterations should be conducted to minimize the structural thicknesses while satisfying the pavement performance prediction for the design life. Optimal design and then one failure iteration must be submitted for review for each design. If the optimization functionality in the software is used then a screenshot of the optimization results must be submitted with the optimized result. The optimization option only optimizes the PCCP thickness, so separate optimized designs should be created for varying joint spacing or dowel bar sizes. A new rigid-pavement trial should be started by first creating a template identifying all subgrade and pavement layers.

1. **Design Features.** The design feature inputs in the software should be based on the INDOT *Standard Specifications* and *Standard Drawings*.

   a. Surface Short-Wave Absorptivity, which is the concrete surface absorptivity value from sunlight. The typical value is 0.85.

   b. Joint Spacing. This is the parameter in the design that determines the pavement performance predictions as a requirement for the design. A typical range for the joint spacing is 15 to 18 ft. The most economical design has the longest joint spacing that satisfies the performance criteria. Ramps pose a unique challenge due to the potential for shorter joint spacing than slab width.

   c. Sealant Type. Use the most predominant transverse sealant type, which is silicone sealant for the transverse joints.
d. Random Joint Spacing. This technique is no longer used.

e. Doweled Transverse Joint. This option should always be selected, except for thin PCCP overlays less than 7”.

f. Dowel Diameter. These should be as specified by the designer. It is not recommended to use 1½” dowel diameter for PCCP less than or equal to 9” thick. All other combinations may be tried. The dowel diameter is limited to 1”, 1 ¼”, and 1 ½”.

g. Dowel Bar Spacing. Dowel bar spacing must be 12in. See Standard Drawings.

h. Widened Slab. This option should be used if the pavement slab is wider than the travel lane width, such as 14 ft and striped at a 12-ft width. This is typically used on divided highways without tied shoulders. For monolithically poured PCC pavement with concrete shoulder, the maximum slab width between longitudinal joint and the pavement edge or between two longitudinal joints should not exceed 14 ft. The slab width should be input into the software.

i. Tied PCC Shoulder or Curb. This is a project-specific input. For a curb with a minimum 2-ft tie width, analyze in the software as “tied shoulder” if the curb is tied to the adjacent concrete pavement.

j. Long Term LTE (Load Transfer Efficiency). This should be 50% to 70% for a sawed longitudinal joint with tie bar, 30% to 50% for a longitudinal construction joint with tie bar, or 0% for no tie bar. Use 60% as the typical value. For curb LTE use 50%.

k. PCC-Base Interface. Unless specified otherwise, the interface is full-friction contact.

l. Erodebility Index. The value for a drainable No. 8 granular base is Very Erosion Resistant (2). The value for a No. 53 granular base is Erosion Resistant (3). The value for cement or asphalt stabilized base is Extremely Resistant (1).

m. Loss of Full Friction. Use 600 as the typical value (50-year life cycle times 12 months per year).
2. **PCC Material Properties.** This section requires inputs for the thickness, thermal and mix properties of concrete.

   a. **Layer Thickness.** This is the thickness of the concrete layer for trial design. The pavement designer should select a trial design thickness based on experience from past projects. The minimum thickness shall be 8”. The selected trial design thickness should be in 0.5-in. increments. If pavement design differs from the typical sections shown in this chapter, approval is required from the Pavement Engineering Manager. Include a sketch or modified drawing with the pavement design and justification for the change. If optimization is used the range of analysis must also be presented. A screen shot of the optimization panel works as sufficient documentation.

   b. **Unit Weight.** This is the dry unit weight of concrete based on AASHTO T 121. A typical value is 145 lb/ft³.

   c. **Poisson Ratio.** This is based on samples tested using AASHTO C 469. A typical value is 0.20.

   d. **Coefficient of Thermal Expansion (CTE).** This is the coefficient based on samples tested using AASHTO TP 60 (for AASHTOWare Pavement ME Design software). A typical range of Indiana CTE for the concrete mix varies from 4.7 x 10⁻⁶ to 6.1 x 10⁻⁶. A typical value for concrete mix is 5.4 x 10⁻⁶.

   e. **Thermal Conductivity.** This is for concrete samples tested using ASTM E 1952. A typical value is 1.25 BTU/h-ft⁻°F.

   f. **Heat Capacity.** This is for concrete samples tested using ASTM D 2766. A typical value is 0.28 BTU/lb⁻°F.

   g. **Cement Type.** Select the appropriate cement type that is expected to be used in the project. A recommended selection is Type I cement.

   h. **Cementitious Material Content.** This value is from a typical mix design in accordance with the INDOT *Standard Specifications*. A typical value of Portland cement content is 400 lb/yd³, the minimum value per *Standard Specifications*. The minimum total cementitious is typically 510 lb/yd³ which includes supplemental cementitious materials such as fly ash.
i. Water/Cement Ratio. This value is from a typical mix design from a previous concrete pavement project in accordance with the INDOT Standard Specifications. A typical value is 0.42 including supplemental cementitious materials such as fly ash.

j. Aggregate Type. Select the type of aggregate to be potentially used in the project limits. Limestone is the most common type of aggregate.

k. PCC Zero-Stress Temperature. This value depends on the Mean Monthly Temperature and Cement content to determine the temperature of PCC zero stress during concrete hardening. The software provides a calculator option to calculate the typical temperature which should be set to True.

l. Ultimate Shrinkage at 40% Relative Humidity. This value represents the ultimate shrinkage of a concrete sample in 40% relative humidity. The typical value is 483 microstrain.

m. Reversible Shrinkage. This value represents the reversible shrinkage of a concrete sample as a percentage of the ultimate shrinkage. The typical value is 50%.

n. Time to Develop 50% of Ultimate Shrinkage. This value represents the time to develop 50% of the ultimate shrinkage. The typical value is 35 days.

o. Curing Method. This option depends on the curing method during construction of the concrete pavement. The typical option is Curing Compound.

p. PCC Strength Properties. This section requires input based on the hierarchical inputs from Level 3 (lowest) to Level 1 (highest). The strength input for Level 3 is Modulus of Rupture. The value for 28-day Modulus of Rupture is 700 psi. The box should not be checked because the software calculates the elastic modulus internally.

304-14.04 Non-Stabilized Base Pavement Layer Design

1. Non-Stabilized – Drainage Layer. The purpose of this pavement layer is to move water from underneath the pavement surface and to provide a platform on which to construct the subsequent PCCP. The typical material for this layer is coarse aggregate No. 8.
a. Non-Stabilized Material. This input is related to the type of granular material to be used in the drainage layer. The typical option is crushed stone, although other materials also can be utilized based on geographical area and availability.

b. Coefficient of Lateral Pressure, $K_0$. This is for a representative sample of crushed stone. The typical value is 0.5.

c. Thickness. This is the thickness of the drainable granular layer only. The typical value is 3 in.

d. Poisson ratio. This is for a representative sample of crushed stone. The typical value is 0.35.

e. Resilient Modulus.

1) Input Level. This option requires input based on the hierarchical inputs from Level 3 (lowest) to Level 1 (highest). The typical value is Level 2.

2) Analysis Type – Annual Representative Value. This is an option to select a representative sample value for the resilient modulus in this unbound layer. Select this option and input the value shown.

3) Material Property – Resilient Modulus. This is the average resilient modulus for the expected in-place granular material based on representative samples. The typical value for Level 2 is 25,000 psi.

2. Non-Stabilized Base – Separation Layer. The purpose of this pavement layer is to provide a separation layer, typically compacted aggregate No. 53, between the drainage layer and the subgrade soil to eliminate migration of fine particles from the subgrade soil if the subbase includes a drainage layer, coarse aggregate No. 8, or to seal off the subgrade from moisture if using a dense graded subbase.

a. Non-Stabilized Material. This input is related to the type of granular material to be used in the separation layer. The typical option is crushed stone.

b. Coefficient of Lateral Pressure, $K_0$. This is for a representative sample of crushed stone. The typical value is 0.5.
c. Thickness. This is the thickness of the separation layer only. The typical thickness is 6 in.

d. Poisson Ratio. This is for a representative sample of crushed stone. The typical value is 0.35.

e. Resilient Modulus.

1) Input Level. This option requires input based on the hierarchical inputs from Level 3 (lowest) to Level 1 (highest). The typical value is Level 2.

2) Analysis Type – Annual Representative Value. This is an option to select a representative sample value for the resilient modulus in this unbound layer. Select this option and input the value shown.

3) Material Property – Resilient Modulus. This is the average resilient modulus for the expected in-place granular material based on representative samples. The typical value for Level 2 is 30,000 psi. There should be no adjustment of the modulus due to the addition of geotextile or geogrid in the soil, subbase, or base layers.

f. Gradation and other engineering properties. The check box indicating that this layer is compacted shall be checked.

304-14.05 Subgrade Treatment Layer Design

1. Subgrade Material – Prepared Subgrade Layer and Natural Subgrade Layer. These pavement layers are the bottom layers in the MEPDG pavement design. The function of these layers is to provide a foundation for the subsequent pavement layers. The natural subgrade layer is the untreated in-situ material beneath the fill and subgrade treatment. For chemically modified materials, such as lime or cement modification, these materials are not classified as stabilized materials in the MEPDG, rather such materials are treated as compacted subgrade soil with increased modulus values that comes from the Geotechnical Report. If an A-7-6 soil is lime modified, it shall be entered as an A-6 in the prepared subgrade layer to approximate the chemical change. The following are the guidelines for inputs in the AASHTOWare Pavement ME Design software.
a. Subgrade Material. This option is related to the type of expected soil in the project limits. The type of soil can be obtained from the Geotechnical Investigation Report. The typical type is based on AASHTO classification.

b. Coefficient of Lateral Pressure, $K_o$. This is for a representative sample of crushed stone. The typical value is 0.5.

c. Thickness. For compacted subgrade layer the input value should be in accordance with the Geotechnical Report recommendation and the corresponding section in INDOT *Standard Specifications*. The natural subgrade is always the bottom most layer and shown with infinite thickness regardless of any entry for thickness.

d. Poisson Ratio. This is for a representative sample of crushed stone. The typical value is 0.35.

e. Resilient Modulus. For seasonal input values, input the resilient modulus values of the soil, Optimum Moisture Content (OMC) for Summer and Fall, 1.2 OMC $M_r$ value for Winter, and 0.8 OMC $M_r$ value for Spring.

1) Input Level. This option requires input based on the hierarchical inputs from Level 3 (lowest) to Level 1 (highest). If using data from Geotechnical Report, use Level 2 first. If data is not available then use Level 3 as default.

2) Analysis Type – Annual Representative Value. This is an option to select a representative sample value for the resilient modulus in this subgrade layer. Select this option and input the value shown.

3) Material Property – Resilient Modulus. This is the average resilient modulus for the expected in-place subgrade soil based on representative samples. The default values for Level 3 are based on the selection of the AASHTO or USCS soil classification. By selecting the type of soil, the software will provide a default value. For compacted subgrade and natural subgrade layers use Level 2 input, and use the values from the Geotechnical Report. There should be no adjustment of the modulus due to addition of geotextile or geogrid in the soil, subbase, or base layers.

f. Gradation and other engineering properties. If the Geotechnical Report is available, the gradation of the given soil type at the project shall be used. Also the check box indicating that this layer is compacted shall be checked for prepared subgrade.
304-14.06 Aggregate Subgrade Treatment Layer Design

Non-Stabilized Base. If the Geotechnical Report specifically calls for an aggregate-only prepared subgrade treatment, then the layer is input as a crushed stone but with the modulus from the Geotechnical Report. The check box indicating that the layer is compacted in the Gradation & Other Engineering Properties shall be checked.

304-14.07 Chemically Stabilized Pavement Layer Design

1. Chemical Stabilization. This type of stabilization is considered stabilized base in the AASHTOWare Pavement ME Design software. INDOT specifies this as chemical modification which should not be confused with the chemical modification described in the prepared subgrade treatment in Section 304-14.05. However, it is not typically used unless it is specifically required by the Geotechnical Report. The purpose of this pavement layer is to support the subsequent upper pavement structural layers. The strength and properties of the stabilized materials are based on the mix design of the materials. For moderately stabilized materials, such as cement or asphalt stabilizations, these materials should be tested for their strength and physical properties. See INDOT Standard Specifications, Section 215.

The following are the guidelines for inputs in the software and are for information purposes only.

a. Material Type. This option is related to the type of expected stabilized material to be used in the project.

b. Layer Thickness. This is the thickness of the stabilized material only, as it is based on trial design. A preliminary value is based on the Geotechnical Report.

c. Unit Weight. This is the unit weight of the stabilized material. This value is a result of the laboratory testing.

d. Elastic/Resilient Modulus. This is the modulus of elasticity or the resilient modulus of the stabilized material and specified in the Geotechnical Report.
e. Thermal Conductivity. This is the thermal conductivity of stabilized material. This value is a result of the laboratory testing.

f. Heat Capacity. This is the heat capacity of the stabilized material. This value is a result of the laboratory testing.

2. Stabilized Drainage Layer for Concrete Pavements. Stabilized drainage layers are not normally specified. The purpose of this pavement layer is to provide a durable pavement drainage layer to remove water from the body of the pavement. The strength and properties of the stabilized materials are based on the mix design of the materials. For fully-stabilized materials such as cement or asphalt stabilizations, these materials should be tested for their strength and physical properties. For a cement-stabilized drainage layer, the following are the guidelines for inputs. The designer is responsible for justifying all of the follow inputs.

a. Material Type. This is the cement stabilized type only.

b. Layer Thickness. This is the thickness of the cement stabilized drainage layer.

c. Unit Weight. This is the unit weight of the cement stabilized drainage layer. This value is a result of the laboratory testing.

d. Elastic/Resilient Modulus. This is the modulus of elasticity or the resilient modulus of the cement-stabilized drainage layer.

e. Modulus of Rupture. This is the flexural strength from the mix design. This value is obtained from the trial batch.

f. Thermal Conductivity. This is the thermal conductivity of stabilized material.

g. Heat Capacity. This is the heat capacity of the stabilized material.

For an asphalt-stabilized drainage layer, the following are the guidelines for the software inputs. An asphalt-stabilized drainage layer would need to be entered as a flexible layer, not as a chemically stabilized layer. This layer would have the HMA properties of an open graded HMA layer. These values are the results of the laboratory testing. The designer is responsible for justifying all of the HMA inputs.
304-14.08 Overlay Design

An overlay pavement system will be constructed on:

1. Flexible (HMA);
2. Rigid (PCCP); or
3. Composite (typically HMA over PCCP).

The existing pavement needs to be evaluated structurally before designing the pavement rehabilitation. A final pavement structure design using the AASHTOWare Pavement ME Design software should include all of the layers shown in the typical sections shown in the series of Figures 304-21. Iterations should be conducted to minimize the overlay thickness while satisfying the pavement performance prediction for the design life.

An HMA overlay should be treated as a flexible pavement structure design with the design performed using the specific inputs for the District within which the project resides, the PG grade, and the mixture type being used on each layer. This data is available to an INDOT designer within the Citrix drive location for the software with a shortcut to it under the user name of each designer. The information is also available to a pavement designer outside of INDOT on the Department’s website.

General Instructions to the Designer in Overlay Design Analysis

1. For an asphalt overlay over an existing asphalt layer on an existing severely cracked concrete layer, the existing concrete layer should be defined as a non-stabilized crushed stone layer. The Resilient Modulus of the crushed stone layer should be determined from FWD test using the average elastic modulus value. There are times when this shows an AC only permanent deformation that is not witnessed in the existing pavements. In this case, it should be looked at as existing concrete with new HMA representing the existing HMA with the existing pavement properties, but that must be proven to be necessary. Typical modulus range for severely cracked concrete pavement varies from 40,000 to 200,000 psi. For the Resilient Modulus input for PCCP Cracking and Seating use 500,000 psi, and for PCCP Rubblization use 50,000 psi.

2. Date of Existing Pavement Construction is the date of construction of the existing layer at the bottom of the proposed mill.
3. The Level chosen is based on how much data is available. Required inputs for each Flexible Rehabilitation Level as follows:
   
a. Rehabilitation Level 1. milled thickness, interface condition, rut (existing AC) through pavement condition survey, and FWD, e.g., modulus, frequency (30 Hz), and temperature (77°F).
b. Rehabilitation Level 2, milled thickness, interface condition, rut (existing AC) through pavement condition survey, and cracking (%).
c. Rehabilitation Level 3, milled thickness, interface condition, and pavement rating through pavement condition survey, e.g., excellent, good, fair, poor, or very poor.

4. All interfaces should be fully bonded, i.e., full friction interface value = 1.

5. A FWD test should be requested on the existing pavement.

6. The reflective cracking performance is presented in a chart of total cracking, but does not influence MEPDG overlay design.

7. Input Level 3 should be used for asphalt material properties of existing asphalt layer.

8. Existing asphalt layers should be combined and input as one existing intermediate or base layer depending upon the thickness of the existing pavement.

9. In existing asphalt layer, if measured aggregate gradation in mix is not available, use recommended gradation information found in the file asphalt concrete (existing).xls, available to an INDOT designer within the Citrix drive location for AASHTOWare Pavement ME Design software with a shortcut to it under the user name of each designer. The information is also available to a pavement designer outside of INDOT on the Department’s website. Cumulative percent passing should be used for aggregate gradation inputs with ¾-in, 3/8-in, No. 4 and No. 200 sieve sizes.

10. SuperPave Performance Grade (PG) should be used for asphalt binder inputs of existing asphalt layer constructed after 1996. Prior to that, asphaltic cement grade (AC) should be used.

11. For unbonded JCP over existing composite pavement, remove all existing asphalt overlays and place a minimum 1-in. bond breaker HMA layer on existing concrete. All existing pavement layers must be structurally analyzed before selecting the pavement treatment. For PCCP over existing HMA pavement, profile milling is needed to smooth out the
surface of the existing HMA pavement. Model the pavement cross section as JPCP on existing asphalt pavement. These existing asphalt pavement properties have to follow the cross sectional properties in the original pavement. Prior to 1996 use AC properties and after 1996 use SuperPave.

PCCP overlay over existing full depth HMA, minimum of 7 in. for 15 year design life. Or PCCP overlay over composite pavement with low truck traffic as per the following table:

<table>
<thead>
<tr>
<th>Two Way AADTT</th>
<th>Thickness</th>
<th>Joint Spacing*</th>
</tr>
</thead>
<tbody>
<tr>
<td>125</td>
<td>6 in.</td>
<td>4 - 6 ft</td>
</tr>
<tr>
<td>100</td>
<td>5 in.</td>
<td>4 - 6 ft</td>
</tr>
<tr>
<td>55</td>
<td>4 in.</td>
<td>4 - 6 ft</td>
</tr>
</tbody>
</table>

*Longitudinal and transverse, no dowel bars in joints

12. When entering the non-destructive testing (NDT) information about an existing HMA layer, the average resilient modulus from the FWD report should be used. Do not subtract one standard deviation.

13. The resilient modulus of the subgrade layers should be taken from the FWD report and be equal to the average modulus of resilience of the subgrade soil. Do not subtract one standard deviation. The two subgrade layers required below the pavement should have the same value for the modulus.

14. When performing an overlay on existing JPCP, the Foundation Support input is required. The modulus of subgrade reaction is measured if you have an FWD report. Enter the number of the month, e.g., 5 for May, 9 for August, etc. in which the FWD was completed. This is found on the first page of the FWD report. The modulus of subgrade reaction is called the Dynamic K-Value of Pavement Support in the FWD report. The average value should be entered. Do not subtract one standard deviation.

15. Crack spacing on existing cracked CRCP INDOT pavements is generally 24 in.

16. If pavement design differs from the typical sections shown in this chapter, include a sketch or modified drawing with the pavement design and justification for the change for the design exception.

17. For PCC overlay over PCC pavement, the existing PCC pavement must be structurally analyzed when selecting the pavement. The existing PCC pavement should be structurally sound or the localized pavement deteriorated areas must be repaired first in order to avoid
future premature deteriorations of the new PCC overlay. A minimum 1-in. HMA bond breaker layer must be placed over the existing PCC pavement prior to placing the PCC overlay.

PCCP overlay over existing PCC pavement in low truck traffic, limited to pavement overlay thickness of 4 to 6 in. and joint spacing of 4 to 6 ft.

<table>
<thead>
<tr>
<th>Remaining Life (%) from FWD</th>
<th>Condition Factor (CF)</th>
<th>Overlay Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0.78</td>
<td>$D_O$</td>
</tr>
<tr>
<td>50</td>
<td>0.89</td>
<td>$D_O$</td>
</tr>
<tr>
<td>75</td>
<td>0.95</td>
<td>$D_O$</td>
</tr>
</tbody>
</table>

$$D_O = \sqrt{D_f^2 - (CF \times D_e)^2}$$

$D_f$ = Thickness of new PCC design for future traffic

$D_e$ = Thickness of existing pavement

Use the same subbase and subgrade properties from FWD results of the existing pavement.

304-15.0 HMA PAVEMENTS AND PAY ITEMS

The INDOT Standard Specifications Section 401 QC/QA-HMA pay item should use the format as follows:

QC/QA-HMA, ______, ______, ______, ______ mm

(ESAL (PG (Course) (Mixture Category) Binder) Designation)

The ESAL categories are listed in Figure 304-15A, ESAL Category for QC/QA-HMA Mixtures. Category 1 shall not be used on INDOT projects, except for shoulders on a case-by-case basis.

EXAMPLE: The pay item QC/QA-HMA, 4, 76, Surface, 9.5 mm represents a QC/QA-HMA-mixture with between 10,000,000 and 30,000,000 ESALs, a PG 76 high-temperature binder, a Surface course, and a mixture designation size of 9.5 mm.

The project designer should use the pay-item descriptions shown in the INDOT Standard Specifications for QC/QA-HMA mixtures.
Section 402 HMA is specified for a small-quantities project or where construction constraints require that the material be placed in narrow widths, non-uniform widths, or drive approaches where density and samples are difficult to obtain. No Section 402 HMA will be specified on mainline or shoulders when the original contract pay item quantities are greater than or equal to 300 tons.

For miscellaneous mixtures (Section 402) such as HMA Rumble Strips, HMA for Approaches, HMA for Temporary Pavement, Widening with HMA, the project designer should specify the applicable pay item and mixtures as listed in the INDOT Standard Specifications.

An HMA pay item should use the following format:

Widening with HMA, Type _______
(Type)

The mixture type is determined from ESALs calculated for the project’s pavement design. The type categories for HMA mixtures are listed in Figure 304-15B, Mixture Type for HMA Mixtures.

EXAMPLE: The pay item HMA Patching, Type B represents a HMA mixture for the range of 0.3 million ≤ ESAL < 3 million, and Patching.

The project designer should use the pay item descriptions shown for HMA mixtures in the INDOT Standard Specifications, Section 401.

304-15.01 Performance Grade (PG) Binder

Performance Graded (PG) Binders for QC/QA mixtures are designed based on their performance-related properties determined for the project’s climate (temperature) and location within the pavement structure. A program developed by FHWA’s Long-Term Pavement Performance (LTPP) program, LTPPBinder ™ should be used to select the grades of the PG Binder for a specific project. Information related to this software is available at http://www.fhwa.dot.gov/pavement/ltpp/ltppbinder.cfm.

Base mixtures are designed for a lower high-temperature than the surface and intermediate mixtures. The PG binder is determined using the LTPP Bind program with inputs based on the speed and amount of traffic for the project. For an HMA overlay, the type and condition of the existing pavement should also be considered. For intersections and roundabouts with traffic of more than 3 million ESALs, it is recommended to bump the PG binder grade one grade up, e.g., if LTPPBIND determines PG 64, then use PG 70.
PG binders are identified with high and low Celsius temperature values. For example, PG 70-22 identifies 70°C as the high-temperature design value and -22°C as the low-temperature design value. The high-temperature value is the average seven-day maximum pavement temperature. The low-temperature value is the lowest air temperature recorded at the weather station(s) nearest the project site.

The binder selection reliability is used to indicate the probability that the design high and low temperatures will not be exceeded during the design life. A value of 64, 70, or 76 should be used for the high-temperature design and a value of -22 will be used for the low-temperature design. The value selected for design high temperature should be evaluated for 98% reliability. However, a design high-temperature value satisfying 90% reliability may be considered for a Traffic Group A roadway (TWRG).

The PG binder for a QC/QA project will be identified in the pay item designation in accordance with INDOT Standard Specifications.

304-15.02 HMA Shoulders

For an HMA paved shoulder of 4 ft or narrower, the project designer should specify the same HMA pay item designations and thicknesses as those used for the adjacent travel lane. For an HMA paved shoulder wider than 4 ft, the project designer should specify the thicknesses and HMA pay item designations for the appropriate typical section identified in the series of Figures 304-21.

Shoulder corrugations should be in accordance with INDOT Standard Specifications Section 302.

304-15.03 HMA for Approaches

HMA for Approaches is a mixture designated for a drive, public-road approach, crossover, turn lane, acceleration or deceleration lane, mailbox approach on a non-paved shoulder, etc. It should be used for a short project where the HMA quantity is less than 300 tons and where the paving involves a large amount of handwork or non-paving movement of the paver and rollers. The limits and the pavement section for HMA for Approaches are shown in the INDOT Standard Drawings for drives, public-road approaches, and crossovers. Where the AADT exceeds the amount shown on the Standard Drawings, the HMA pavement section must be determined in accordance with Section 304-14. See INDOT Standard Specifications, Section 610 and 611.
For a public-road approach, the limits for HMA mixtures for approaches may be extended to include up to an additional 100 ft of pavement to satisfy project requirements. If the project requires more than 100 ft of additional pavement the entire public approach section will be designed based on MEPDG.

For an HMA turn lane, HMA acceleration or deceleration lane, HMA wedges for a bridge deck overlay project, or HMA approaches for a bridge-replacement project that require less than 300 tons of HMA material, the pavement will be designed in accordance with Section 304-14 as HMA for Approaches of the type required.

For a mailbox approach on a non-stabilized shoulder, HMA for Approaches of the type required, should be used as specified on the INDOT Standard Drawings.

**304-15.04 Composite Pavement Rehabilitation**

HMA over asphalt and PCC composite pavement will be designed to match the existing pavement. If there is existing excessive reflective cracking, the designer needs to obtain enough information to determine where partial depth patching and full depth patching is required. FWD is recommended on composite pavements to determine the structural integrity and the need for undersealing. The longitudinal joint of the widened composite pavement should not be placed in a wheel path of a travel lane.

If the existing pavement has open-graded subbase with underdrains, the existing longitudinal underdrain system will be perpetuated with additional outlets added in accordance with Section 304-18. If the existing pavement has dense-graded subbase, underdrains are typically not added. The existing asphalt over PCC composite pavement should be milled in accordance with Section 304-10, and prepared in accordance with the INDOT Standard Specifications, Section 306.

**304-16.0 PCC PAVEMENT AND PAY ITEMS**

The requirements for Portland Cement Concrete Pavement (PCCP) are given in the INDOT Standard Specifications, Section 500. PCCP is constructed on subbase for PCCP (drainable subbase), or dense graded subbase on a prepared subgrade.

The subgrade should be designed in accordance with the Geotechnical Report. The geotechnical recommendations may include a soil modification or stabilization process, subgrade-treatment type, or a compacted-aggregate stabilization layer.
Subbase for PCCP is placed on the prepared subgrade and is composed of 3 in. of coarse aggregate No. 8 on 6 in. of compacted aggregate No. 53. The coarse aggregate No. 8 is a permeable layer that collects and removes water entering the pavement subbase system. The compacted aggregate No. 53 is a dense layer that separates the subgrade from water entering the pavement subbase system. Underdrains must be included where Subbase for PCCP is specified. Dense Graded Subbase is used under miscellaneous PCCP such as a drive, reinforced-concrete bridge approach, etc., or may be used where underdrains are not warranted although in the case where Dense Graded Subbase is used the design life may be reduced to 20 years. Dense Graded Subbase is composed of 6 in. of compacted aggregate No. 53, but is paid for as “Subgrade Treatment, Type IIIA” under driveway pavement.

The designed thickness of PCCP determined by the AASHTOware Pavement ME Design software is placed on Subbase for PCCP. The minimum PCCP thickness is 7 in. The transverse joint spacing and dowel bar diameter in the concrete pavement joints are designed in accordance with the MEPDG and are constructed as contraction joints type D-1. The joint spacing should be shortened where necessary to meet a drive, inlet, adjacent lane, etc., so that all joints are continuous across the entire width of pavement including shoulders. The additional D-1 joints should be included in the contract quantities.

Non-standard joints are not to be used in a pavement without approval of the Department’s Director of Pavement Division. If a project designer desires to utilize non-standard pavement joints in an individual contract, a submittal should be made to the Director of Pavement Division.

Quality Control/Quality Assurance (QC/QA) PCCP pay items and PCCP pay items, as described in the INDOT Standard Specifications, Section 500 are used for a project specifying PCCP. The criteria for using QC/QA-PCCP or PCCP are based on the area of PCC pavement specified. For a project PCCP quantity of 7200 yd² (one lot) or greater, the pay item should be “QC/QA-PCCP, ___ in.” For a project PCCP quantity of less than 7200 yd², the pay item should be “PCCP, ___ in.”

304-16.01 PCCP Preventive Maintenance Treatment (CPR)

The pavement work on a PCCP overlay project may include milling of the existing pavement, the placement of an overlay, or a combination of these elements. Concrete Pavement Restoration (CPR), of a PCCP may be used where the existing PCCP is considered to be structurally sufficient but has reduced serviceability. CPR alternatives are full or partial depth patching, PCCP profiling, joint resealing, retrofit load transfer, shoulder restoration, slab stabilization (undersealing), longitudinal crack and joint repair, or combinations of these alternatives.
The condition of the driving lane of PCCP is an indicator of the project’s suitability for CPR. FWD testing and core investigation for “D” cracking at joints should be completed. A PCCP investigation where cores indicate a “D” cracking distress is not a candidate for CPR or PM treatment.

Undersealing consists of a localized activity where a fluid material is pumped under the concrete slab to add support and to fill voids under the pavement. PCCP or asphalt over PCC composite pavement should be tested with a FWD as described in Section 304-13.01 to determine size and limits of voids underlying the pavement. PCCP on open graded subbase should not be undersealed. In urban areas utilities may be an issue that prohibits undersealing as this will fill old cracked storm drainage systems and may damage other buried utilities. Pavement designers need to ascertain whether utilities can be avoided during an undersealing operation in urban areas.

The cost of the recommended rehabilitation should be compared to the cost of replacing the existing pavement or an alternate rehabilitation technique using life-cycle cost analysis.

**304-16.02 Continuously Reinforced Concrete Pavement (CRCP)**

**304-16.02(01) CRCP Design**

CRCP has the potential to provide a long-term, zero-maintenance, service life under heavy traffic loadings and challenging environmental conditions, provided proper design and quality construction practices are utilized. See FHWA–HIF-12-039 TechBrief. The AASHTOWare Pavement ME Design software (formerly DARWinME) should be utilized to design a CRCP. The best source for a national overview of CRCP performance is the Long-Term Pavement Performance (LTPP) Program. General Pavement Study (GPS) 5, included 85 CRCP experimental sections in 29 states.

CRCP differs from other concrete pavements as follows:

1. CRCP has no active transverse contraction joints, except at ends.

2. Continuous longitudinal reinforcement is provided that results in tight cracks in the concrete at about 2-ft to 8-ft spacing. Sufficient reinforcement is necessary to keep the cracks tight.

3. CRCP can extend, joint free, for many miles with breaks provided only at structures, such as bridges.
CRCP design focuses on managing the cracking that develops so as to reduce the structural distress that may develop as a result of traffic and environmental loadings. These distresses include punchouts, steel rupture, and crack spalling.

CRCP design involves determining the proper combination of the following:

1. slab thickness;
2. concrete mixture constituents and properties; and
3. steel reinforcement content and location (critical element).

Other important features that a designer must require for a good CRCP design are as follows:

1. provide for sufficient slab edge support (critical element);
2. strengthen or treatment of the subgrade; and
3. provide non-erodible bases that also provide friction that leads to desirable transverse cracking patterns.

Most transverse cracks form at very early ages before a pavement is open to traffic, and cracking may continue for several years after concrete placement. Transverse cracks occur when and where the tensile stress, due to the restrained volume changes in the concrete, exceeds the concrete’s developing tensile strength. New transverse cracks occur roughly at the midpoint between two previously formed cracks, where the maximum concrete stress occurs. Crack formation continues until concrete strength exceeds the stresses due to the restrained volume change. Recognizing that the tensile strength of the concrete and the tensile stresses vary along the length of the slab, the transverse crack spacing pattern is never uniform, but the majority of cracks should be spaced within the desired range of 2 ft to 8 ft.

304-16.02(02) CRCP Reinforcement

The use of longitudinal steel reinforcement, typically Grade 60 bars, results in a series of closely spaced transverse cracks. The steel reinforcement is used to control cracking and is spaced, typically between 2 ft and 8 ft, and the amount of opening at the cracks and to maintain high levels of load transfer across them. CRCP should be built with longitudinal reinforcing steel percentages in the range of 0.65 to 0.80 percent; lower in milder climates and higher in harsher climates. Equally important as the percentage of steel content is the bond area between the concrete and the bars, which the Federal Highway Administration recommends at a minimum of 0.030 square inch per cubic inch of concrete (FHWA 1990). Design steel content provides a balance between crack width (< 0.02 in. at surface over design life), crack spacing, and crack load transfer capability.
Vertical placement of the bars also affects performance—placed too high, the bars may corrode due to inadequate cover; placed too low, the bars are too far away to keep the cracks tight at the surface. The common position of the reinforcement is between \( \frac{1}{3} \) and \( \frac{1}{2} \) the slab thickness measured from the pavement surface (CRSI 2009).

Steel bars normally come in standard lengths of 60 ft and must be lap-spliced to form a continuous longitudinal mat. The lap-splicing patterns should be staggered. Transverse bars are always used to support longitudinal reinforcement. The transverse bars are placed on bar supports. The bars also keep tight any longitudinal cracking that may develop. The vertical position of the bars is set by the supports and diameters of the transverse and the longitudinal bars, and the tolerance is usually up 0.50 in. and down 1.0 in. The chairs or bar supports must be stable and should not sink into the base prior to paving. Horizontal spacing tolerances are less stringent, but it is important that longitudinal bar placement does not impede placement or consolidation of the concrete. The horizontal bar spacing tolerance should be 1.0 in.

The development of continuous bar supports, commonly known as transverse bar assemblies (TBAs), has led to speedier placement of the steel mat. A TBA is a transverse bar to which are welded steel supports which serve as chairs, and U-shaped clips. The spacing of the clips along the bar matches that required of the longitudinal bars. When the longitudinal bars are installed into the clips, the clips hold them in position vertically and keep them from moving transversely, while allowing a bit of longitudinal movement.
304-16.02(03) CRCP Edge Support/Shoulders

Proper edge support (tied concrete shoulder) adjacent to mainline CRCP reduces wheel load stresses and deflections and the occurrence of punchouts, reduces longitudinal joint maintenance issues; reduces shoulder maintenance needs, and provides support for traffic detours. To be considered a “widened slab”, research indicates that the slab needs to have a minimum width of 13 ft to minimize longitudinal cracking (INDOT uses 14 ft), and be striped to 12 ft to significantly reduce the stresses and deflections due to heavy truck traffic near the pavement edge. Current best practice to improve the edge support is to use a tied-concrete shoulder or a widened outside lane, a “widened slab” with an asphalt shoulder.

304-16.02(04) CRCP End Treatments

Two types of end treatments at structures are used for CRCP:

1. Wide flange beam joint. This treatment serves as an expansion joint and allows the end to move freely as the concrete expands and contracts with changing temperature.

2. Anchor lugs. This treatment, consisting of several lugs below the slab and tied unto the slab end, attempts to restrain any movement from taking place at the ends.

A simple conventional doweled expansion joint may be used as part of the approach slabs at a structure.

304-16.02(05) CRCP Concrete Curing

CRCP can be placed both during the daytime and nighttime hours. Paving at night when daytime temperatures would be very hot has been shown to result in better performing CRCP because the development of heat of hydration and high ambient daytime temperatures due to solar radiation do not coincide. Better temperature specifications and temperature management during paving are leading to better performing CRCP. Specifications limit the concrete temperature to a range of 50° F to 90° F. Other measures to reduce heat may include changing the concrete mixture constituents and proportions for lower heat of hydration, specifying wetting of the base and steel bars just in front of the paver, and whitewashing the asphalt base prior to placement of the reinforcement, as long as it does not reduce bonding and friction with the CRCP, as this will greatly affect crack spacing and width. The use of HIPERPAV® software at the construction site can provide relative information regarding expected CRCP cracking patterns if there are drastic
temperature changes so that various remediation measures, e.g., changes in concrete mixture, curing techniques, etc. can be implemented.

304-17.0 MISCELLANEOUS PAVEMENT PROJECT ELEMENTS

304-17.01 Subgrade

A prepared subgrade is required before construction of the pavement. Subgrade is the upper portion of the natural ground or constructed embankment upon which the pavement structure and shoulders are to be constructed. A geotechnical investigation is required for new pavement, reconstructed pavement, rubblized concrete, cracked and seated concrete, or widening (mainline or shoulder). The geotechnical investigation may not be required for a one- or two-layer overlay, PM, or surface preservation treatment projects. The geotechnical investigation should be requested by the district Pavement Engineer, project manager, or the consultant’s pavement designer or project engineer. A full geotechnical investigation usually takes 120 calendar days. Specified subgrade treatments shall be in accordance with recommendations in the Geotechnical Report and INDOT Standard Specifications Section 207.

304-17.02 Temporary Pavement

Temporary pavement is used for maintenance of traffic (MOT) and will take the form of:

1. temporary cross-over;
2. temporary run-around, see Standard Drawings 713-TCTR-01 and -04;
3. temporary widening (auxiliary lane); or
4. temporary ramp.

Temporary pavement should be designed using AASHTOWare Pavement ME Design software considering 95% reliability and minimum 2 construction seasons or as specified in the pavement design request. The temporary pavement should be designed with the proper geometric and design speed considerations, i.e., cross slope, superelevation, profile grade, etc. The subgrade should be prepared based on Geotechnical Report recommendation. If the Geotechnical Report is not available, use Resilient Modulus of 3000 psi for prepared subgrade, 1500 psi for the natural subgrade and Type IIIA subgrade treatment. If a temporary pavement is to be used as a permanent shoulder, pay items should not be temporary pavement, but should be QC/QA HMA.
304-17.03 Driveways

*Standard Drawings* E 610-DRIV-01 through -07.

304-17.04 U-Turn Median Opening

*Standard Drawing* E 610-UTMO-01.

304-17.05 Public Road Approach

The pavement design for a public-road-approach pavement should be in accordance with the INDOT *Standard Drawings*. An individual pavement recommendation for a public road approach is required only where the AADTT exceed the values listed in the INDOT *Standard Drawings*.

304-17.06 Bridge Deck Overlay

An individual pavement design is not required for a bridge deck overlay project when the maintenance of traffic does not utilize the shoulder. If the shoulder is utilized for maintenance of traffic a pavement design is required. For both cases, the district Pavement Engineer will provide the pavement design recommendations.

304-17.07 Seal Coat

Seal coat, or chip seal, is used to seal a shoulder, to seal a very low-traffic-volume roadway, or during construction to bond loose material to allow construction traffic to use the surface. The requirements for seal coat are shown in the INDOT *Standard Specifications*, Section 404.

304-17.08 Prime Coat

Prime coat is only required for a rubblized base that is to be overlaid. The prime coat binds the top portion of the rubblized base with the first HMA layer so that the HMA material will not slide relative to the base material during compaction of the HMA. Prime coat shall not be specified to be placed on a compacted aggregate before an HMA Base or Intermediate is laid on subgrade. Prime coat should be specified on chemically modified soil or soil compacted to density and moisture requirements before an HMA Base or Intermediate is laid on subgrade. This must be
specified in the Pavement Design Memorandum. The requirements for prime coat are shown in the INDOT Standard Specifications, Section 405.

304-17.09 Tack Coat

Tack coat is required beneath each course of HMA material that is placed on an existing pavement or newly-constructed HMA course. The tack coat binds the new HMA material to the material already in place. HMA or PCCP is to be tacked prior to placement of an HMA mixture. The requirements for tack coat are shown in the INDOT Standard Specifications, Section 406.

304-17.10 Base Seal

Base seal is used to help maintain the integrity of the OG layer. Prior to placing an open-graded mixture, the underlying HMA course (HMA Base) shall have a full width base seal applied in accordance with INDOT Standard Specifications, Section 415. The base seal materials are applied to the pavement surface uniformly with a distributor at a relatively high application rate of $0.22 \pm 0.02$ gal/sq yd. The base seal materials also must cure a minimum of two hours after application before resuming paving operations.

304-17.11 Curbs and Shoulders

PCCP is constructed with curb and gutter sections, integral concrete curbs, a widened outside lane with HMA shoulder, or tied full-depth concrete shoulders. The curb and gutter sections, integral curbs, widened outside lane, or tied shoulders stiffen the outside edge of pavement to reduce deflections. D-1 joints are required across the entire PCCP mainline. Compacted aggregate or geotextile should be specified alongside PCCP curbs or shoulders to prevent erodible material from infiltrating the underdrain system. The typical sections for PCCP shoulders are included in the series of Figures 304-21.

304-17.12 Reinforced-Concrete Bridge Approach (RCBA)

The requirements for an RCBA are shown in the INDOT Standard Specifications, Section 609. The RCBA is constructed on dense-graded subbase on prepared subgrade.

An RCBA is used at a bridge to transition from PCC or HMA pavement to the bridge deck or mudwall. For PCCP, the RCBA spans from the sleeper slab to the pavement ledge on the mudwall.
For HMA pavement, the RCBA spans from the end of the HMA pavement to the pavement ledge on the mudwall. The RCBA is reinforced to account for unsupported conditions due to settlement at the end bent or abutment. The RCBA and reinforcing-steel details are shown on the INDOT Standard Drawings.

304-18.0 UNDERDRAINS

An underdrain is a system of perforated pipe and coarse aggregate installed longitudinally in the vicinity of a pavement edge. The purpose of an underdrain is to remove water from the subgrade and the pavement structure. An Underdrain Table is required in the plans. The designer should consult the Geotechnical Investigation Report for the need for subsurface drains. The pavement designer must determine whether underdrains are a benefit to the life of the pavement and are cost effective. See INDOT Standard Specifications, Section 718.

It is possible that underdrains may not be warranted; however, subsurface drains may be required on a project to remove ground water as required by the Geotechnical Investigation Report.

304-18.01 Definitions

Aggregate for Underdrains. Coarse aggregate No. 8 or coarse aggregate, No. 9 used to backfill an underdrain pipe trench.

Dual-Access Underdrain. A run of underdrain that features outlet pipes connected to both ends of the underdrain pipe. The dual-access outlet pipes are installed to provide access to the underdrain pipe for inspection and maintenance purposes.

Geotextiles for Underdrain. An engineered geotextile fabric used to prevent soil particles from contaminating an underdrain pipe and aggregate for underdrains. See INDOT Standard Specifications, Section 900 for geotextile materials.

HMA for Underdrains. An open-graded HMA used to patch an existing asphalt shoulder over a retrofitted underdrain pipe or an outlet pipe.

Intercept Elevation. The invert elevation at the connection between an underdrain pipe and a PVC connection at a drainage structure or outlet pipe.

Intercept Station. The station at which the connection between an underdrain pipe and a Polyvinyl Chloride (PVC) connection at a drainage structure or outlet pipe occurs.
Obstacle. A project feature, such as a paving exception or bridge, culvert, that prevents the continuous installation of underdrain pipe.

Outlet Elevation. The invert elevation of an outlet pipe or PVC pipe connection where the collected water leaves the outlet pipe.

Outlet Pipe. A non-perforated pipe that conveys water collected by the underdrain pipe to a side ditch, median ditch, or drainage structure. An outlet pipe may also be installed at the high end of an underdrain pipe to create a dual-access underdrain.

Outlet Protector. A concrete slab constructed on a sideslope to protect the outlet-pipe end.

Outlet Station. The station where an outlet pipe discharges to the sideslope or is connected to a drainage structure.

Retrofit Underdrain. An underdrain pipe installed along an existing pavement edge in conjunction with a pavement rehabilitation operation, such as rubblization, cracked and seated, or overlaying.

Rodent Screen. Metal mesh screen fabricated in accordance with the specifications that fits inside the outlet pipe to prevent rodents and debris from entering the underdrain system.

Single-Access Underdrain. A run of underdrain that features an outlet pipe connected to the low end of the underdrain pipe only.

Special Underdrain. An underdrain pipe installed at a specified slope that is not parallel to the pavement profile or a constant depth that differs from that shown in the series of Figures 304-21.

Tangent Grade. The specified grade between two adjacent points of vertical inflection (PVIs) on the vertical alignment of the proposed pavement.

Underdrain Pipe. A perforated pipe installed at the bottom of a longitudinal or transverse underdrain trench.

Underdrain Run. An individual segment of underdrain pipe and its associated outlet pipe or pipes.

Underdrain System. The system that collects water from the subgrade and pavement structure and conveys it to the drainage system. Underdrain-system elements include the underdrain trench, underdrain pipe, aggregate for underdrains, geotextiles for underdrain, outlet pipe, rodent screen, outlet protector etc.
**Video Inspection.** The process of inspecting an individual underdrain run after installation using a video camera. Video inspection can also be performed on existing underdrains to find damaged portions that need repaired.

**304-18.02 Existing Underdrain Perpetuation**

A roadway with existing underdrains should have all outlet pipes perpetuated as part of the work. The project designer should determine if any existing underdrains, longitudinal or transverse, are present, and locate all existing outlet pipes to evaluate them for needed maintenance or repair. Required repair or maintenance, such as unearthing and replacing an outlet pipe or reconstructing an outlet protector, should be included in the proposed work. If there are retro-fit underdrains on a project, the designer should determine if any existing underdrains are present and locate all existing outlet pipes to coordinate them with any new outlets proposed.

**304-18.03 Underdrain Warrants**

Underdrains are required for each project, including a LPA project that satisfies any of the conditions as follows:

1. new pavement or reconstructed pavement with a design-year Average Annual Daily Truck Traffic (AADTT) volume of 100 per day or greater, and a length of at least 1 center lane mile; or

2. the pavement sections adjacent to the project area have existing underdrains; or

3. where specific geotechnical conditions are identified that require subsurface drains as stated in the Geotechnical Investigation Report.

Underdrains are also required when using Subbase for PCCP, HMA class OG mixture QC/QA-HMA 5, 76, Intermediate, OG 19.0 mm, or where an existing PCCP is to be cracked and seated or rubblized.

Underdrains are not typically constructed on a functional or preventive maintenance treatment project, except where existing underdrains are not performing adequately.
**304-18.04 Design Criteria**

Proper design of the underdrains is critical for the life of the pavement. The following items shall be addressed in the design of underdrains.

**304-18.04(01) Slope**

1. **Underdrain Pipe.** Where the tangent grade is 0.2% or steeper, the underdrain pipe will be installed at a fixed depth below the pavement. Where the tangent grade is flatter than 0.2% or if the Geotechnical Report indicates that the underdrain pipe should be installed at a depth other than that shown in the series of Figures 304-21, special underdrains are required. The special underdrain slope should be 0.2% or steeper.

2. **Outlet Pipe.** The flattest outlet pipe slope permitted is 0.3%.

**304-18.04(02) Size**

1. **Underdrain Pipe.** Construction of new pavement requires underdrain pipe of 6-in. diameter. Rehabilitation of existing pavement may utilize underdrain pipe of 4-in. diameter.

2. **Outlet Pipe.** Outlet pipe of 6-in. diameter is required. If underdrain pipe of 4-in. diameter must be used, outlet pipe fittings will be utilized to increase the outlet pipe size to 6-in.

**304-18.04(03) Outlet Spacing**

An outlet pipe is required at the low point of a sag vertical curve. It is also required at other low points encountered along the vertical alignment, such as the project beginning or ending point, or at an obstacle location.

Additional outlet pipes are likely to be required throughout the project limits. The maximum underdrain-pipe length should not exceed 600 ft. If the proposed underdrain-pipe length is greater than 400 ft, a dual-access underdrain is required. If the outlet spacing results in an underdrain-pipe length that is 400 ft or less, a single-access underdrain should be utilized.
**304-18.04(04) Location**

An underdrain, where warranted in accordance with Section 304-18.03, should be constructed along each pavement edge. The underdrain should be continuous through each intersection, ramp, turn lane, taper, etc., and should be located in the pavement section as shown in the series of Figures 304-21. For an approach where an underdrain is warranted in accordance with Section 304-18.03, the underdrain should extend from the mainline underdrain to the limit of the new approach pavement.

1. **Underdrain Pipe.** The underdrain-pipe location within each proposed cross section should be as shown in the series of Figures 304-21.

   If an inlet, catch basin, manhole, or similar structure is located along the alignment of an underdrain pipe, the underdrain pipe may be connected directly to the drainage structure. The connection should be at least 6 in. above the structure invert elevation and a rodent screen should be placed on the outlet end of the underdrain pipe.

   A direct connection of an underdrain pipe to a pipe culvert or a precast-concrete culvert should be avoided.

2. **Outlet Pipe.** The connection between an outlet pipe and an underdrain pipe should be as shown on the INDOT Standard Drawings. 90-deg elbows or tees should not be utilized in these connections.

   One of the 45-deg elbows may be omitted if necessary to provide a satisfactory outlet.

Separate outlet pipes should be provided for each underdrain pipe. Outlet pipes for adjacent underdrain pipes at a sag-vertical-curve low point or for adjacent dual-access underdrains should be installed in a common trench as shown on the INDOT Standard Drawings. Outlet pipes installed in a common trench should outlet at the same elevation.

The outlet elevation should be at least 2 ft above the flowline elevation of a side ditch, 1 ft above the flowline elevation of a median ditch, or 0.5 ft above the invert elevation of an inlet, catch basin, manhole, or similar structure.

If an underdrain pipe has no suitable outlet available at an adjacent ditch line or drainage structure, the outlet pipe may be installed under the pavement to an acceptable outlet on the opposite side of the roadway. The outlet-pipe installation should be designed so as not to conflict with the underdrain-pipe installation along the opposite pavement edge.
304-18.04(05) Backfill

1. **Underdrain Pipe.** Aggregate for underdrains is used to backfill an underdrain pipe trench. A retrofit underdrain requires HMA for underdrains for patching an existing asphalt shoulder above the underdrain-pipe trench as shown on the INDOT Standard Drawings.

2. **Outlet Pipe.** Outlet-pipe backfill includes structure backfill and suitable material placed as shown on the INDOT Standard Drawings. HMA for underdrains is required for patching an existing asphalt shoulder above the outlet-pipe trench associated with a retrofit underdrain as shown on the INDOT Standard Drawings.

304-18.04(06) Outlet Protector

An outlet protector is required at each location where an outlet pipe intersects a median or side slope. An outlet protector may contain two outlet pipes.

The INDOT Standard Drawings series E718-UNDR includes details for each available protector type.

Figure 304-18A, Outlet Protector Slope Limits, includes acceptable slopes for construction of each outlet-protector type.

The outlet protector selected should be the largest protector appropriate for the proposed slope that can be constructed considering all conflicts to the outlet location. Type 1 outlet protectors should typically be used on new alignment projects for the side slope outlets. The smaller Type 2 and Type 3 outlet protectors should only be used for median outlets and in limited applications on these type projects.

304-18.04(07) Geotextiles for Underdrain

There are two applications where geotextiles are used in conjunction with underdrain-pipe installation. The first application is as an underdrain-pipe trench liner. Trench lining should be used only if the Geotechnical Investigation Report recommends such an installation. The second application for geotextile is to prevent the contamination of the underdrain-pipe backfill during the construction of embankment behind a concrete curb. Installation of the geotextile should be as shown in the series of Figures 304-21, and is required in conjunction with curb construction above an underdrain pipe. Geotextile should be required everywhere an open graded drainage material is
in contact with an in situ soil, compacted aggregate shoulder wedge, or other material that has fine grain particles with the ability to infiltrate and clog the drainage layer.

304-18.04(08) Video Inspection

Video inspection of an underdrain system should be included in each new construction as well as rehabilitation project with at least 3,000 ft of underdrain pipe. The contract quantity should be as shown in Figure 304-18B, Video Inspection Contract Quantities.

304-18.05 Contract Document Preparation

304-18.05(01) Plans

Information related to underdrains should be shown on the plans as follows:

1. Typical Cross Sections Sheet.
   a. The underdrain pipe location as illustrated in the series of Figures 304-21.
   b. Underdrain-pipe trench and backfill details.

2. Plan and Profile Sheet. Special-underdrain limits and slopes should be shown on the profile portion of the sheet.

3. Detail Sheets. All project-specific details should be shown on these sheets.

4. Underdrain Table.
   a. Underdrain Pipe.
      1) Beginning and ending stations
      2) Flowline elevations at beginning and ending stations
      3) Pipe size
      4) Special-underdrain grade, if applicable
      5) Pipe quantity
      6) Aggregate for underdrains quantity
      7) HMA for underdrains quantity, if applicable
8) Geotextiles for underdrains quantity, if applicable

b. Outlet Pipe.

1) Outlet station
2) Outlet elevation
3) Intercept station
4) Intercept elevation
5) Outlet protector or structure number at outfall
6) Outlet ditch or drainage structure invert elevation at outfall
7) Pipe quantity
8) Structure-backfill quantity
9) HMA for underdrains quantity

c. Outlet Protectors.

1) Type
2) Location
3) Quantity

304-18.05(02) Specifications

Requirements for underdrains are shown in INDOT Standard Specifications, Sections 700 and 900.

304-18.05(03) Standard Drawings

Details for underdrains and outlet protectors are shown on the INDOT Standard Drawings series E718-UNDR.
304-18.05(04) Pay Items

The designer should determine the contract quantities for the appropriate pay items associated with the underdrain construction. See Chapter 17.

304-19.0 PREVENTIVE MAINTENANCE

Preventive Maintenance (PM) treatments are part of the overall pavement preservation program. A PM project is intended to arrest light deterioration, retard progressive damage, and reduce the need for routine maintenance. A PM treatment typically does not add structural strength to the pavement. The proper time for PM is before the pavement experiences severe distress, structural problems, moisture, or aging-related damage. These activities can be cyclical in nature and may correct minor deficiencies as a secondary benefit. For PM treatment service life, see Figure 304-14A, Pavement Design Life. In considering a PM treatment, the overall program schedule of the pavement section should be considered. To achieve the optimal benefit of the PM treatment, it should not be applied if rehabilitation is planned within the service life of the PM treatment.

A PM treatment is not used where the purpose of the project is to correct pavement cross slope, horizontal alignment, vertical alignment, superelevation problems, placement of a turn lane or auxiliary lane, improvement of public-road approach or drive, or guardrail adjustment or repair. A PM project may include incidental enhancements or combinations at an isolated location in accordance with Chapter 56.

Regardless of the pavement type, proper drainage is essential to the performance of the pavement. Drainage inspection and cleaning consists of the inspection of drainage structures, e.g., underdrain outlets, ditches, catch basins, inlets, and the cleaning of these structures to maintain or restore the flow of water. The locations of underdrains should be identified and the outlets periodically cleaned. The INDOT Field Operations Handbook provides for drainage inspection and cleaning details.

The most commonly used PM treatments are described below. See Figure 304-19A for HMA pavement treatments or Figure 304-19B for PCCP treatments. Further descriptions of available Pavement Preservation Treatments can be found in the INDOT SPR-3114 Treatment Guidelines for Pavement Preservation.

A least cost of ownership analysis as described in Section 304-4.0, should be done for each PM project to determine the most economical treatment.
304-19.01 HMA Pavement PM Treatments

A certain amount of partial-depth or full-depth patching may be required in conjunction with HMA PM Treatments. Partial-depth or full-depth patching will consist of complete removal of a deteriorated section of the HMA pavement and patching it with HMA.

1. **Crack Sealing and Filling.** Crack sealing and filling is the cleaning and sealing or filling of open cracks or joints in asphalt pavement and shoulders to prevent the entry of moisture and debris. The selection of sealing or filling is based on crack movement and crack deterioration. Moving or working cracks, e.g. transverse crack or reflective crack, is defined as an annual crack opening that moves greater than 0.1 in. vertically or horizontally due to thermal expansion and contraction or stress concentration at pavement overlaying joints. Those types of cracks should be considered for crack sealing. Cracks with an annual crack opening with movement of < 0.1 in. or no annual movement, e.g. longitudinal or longitudinal reflective, should be considered for crack filling. Cracks must be clean and dry and may be routed prior to sealing or filling. The major objective of routing is to provide a uniform and smooth edged rectangular reservoir to let the sealant material adhere better with the asphalt pavement and for allowing the sealant level to remain below the pavement surface, which protects the sealant from traffic and snowplow damage. Therefore, routing is strongly recommended for any crack sealing activity as well as crack filling longitudinal joints. This technique may be used for sealing cracks on a newer composite pavement where reflective cracks have developed. This PM treatment may be periodic once more cracks develop as the pavement ages.

Guidelines for selecting a pavement section for crack sealing and filling are as follows:

a. **AADT.** Crack sealing and filling may be performed on any roadway regardless of traffic volume, provided adequate traffic control is provided.

b. **Pavement Distresses.** Crack sealing and filling may be used to correct low to medium severity surface cracks.

c. **Rutting.** Crack sealing and filling does not correct rutting.

d. **Roughness.** Crack sealing and filling does not affect roughness. Roughness is typically not a consideration for crack sealing.

e. **Friction.** Friction is typically not a consideration for crack sealing and filling. However, overband crack sealing may lower the friction number (FN).
f. Surface Aging. Crack sealing and filling does not correct surface aging deficiencies.

2. Fog Sealing. A fog seal is a method of adding asphalt to an existing pavement surface to improve sealing or waterproofing, prevent further stone loss by holding aggregate in place, retarding the age hardening of the asphalt, and improve the surface appearance. However, inappropriate use can result in a slick pavement and tracking of excess material. The pavement section should show no structural deficiencies prior to fog sealing. Fog sealing is generally recommended for shoulders or chip sealed surfaces.

Guidelines for selecting a pavement section for fog sealing are as follows:

a. AADT. Typically less than 5,000. However, fog sealing may be considered on a higher volume road if traffic can be controlled.

b. Pavement Distresses. Low severity environmental-related surface cracks.

c. Rutting. Fog sealing does not correct rutting.

d. Roughness. Fog sealing does not improve roughness.

e. Friction. Fog seal should not be applied to a road with a low FN. Fog seal will reduce FN for a period until the material fully cures.

f. Surface Aging. A fog seal may be used to restore an aged, oxidized, or raveled surface.

g. Longitudinal joint. Fog seal is required on surface layer over longitudinal joint 24-in. in width per Recurring Special Provision 401-R-581.

3. Seal Coat. Seal coat is the treatment of the pavement surface with liquid asphalt material and coarse aggregate to prevent deterioration of the surface. Seal coat is often called chip sealing. It provides waterproofing, low-severity crack sealing, and improved friction. The pavement section should show no structural deficiencies prior to chip sealing. Isolated areas with structural deficiencies shall be repaired prior to chip sealing. A previously seal-coated surface may be sealed again.
Guidelines for selecting a pavement section for seal coat are as follows:

a. **AADT.** Typically used if less than 5,000. A seal coat may be considered on a higher-volume road if traffic can be controlled, i.e. total road closure, extended lane closures. A seal coat may be specified for the shoulders of any road regardless of AADT.

b. **Pavement Distresses.** A seal coat will mitigate low to medium severity surface cracking.

c. **Rutting.** Seal coat does not correct rutting and should not be used where existing ruts are greater than 0.25 in. Seal coating a road with more than 0.25-in. ruts may lead to wheel path flushing.

d. **Roughness.** A seal coat will not improve the International Roughness Index (IRI).

e. **Friction.** A pavement with a low FN may be considered for a chip seal surface treatment. A seal coat will restore surface friction.

f. **Surface Aging.** A seal coat may be used to stop future deterioration of an asphalt pavement due to age hardening, oxidation, or minor raveling.

For mainline pavement with AADT over 1,000, asphalt for seal coat type P should be specified.

The type of seal coat should be specified as follows:

a. **Type 2, 3, 2P or 3P.** These are single-course seal coats appropriate for paved mainline or shoulders. The P designation indicates that polymer modified asphalt is specified.

b. **Type 5, 6, 7, 5P, 6P or 7P.** These are double-course seal coats appropriate for unpaved mainline or unpaved shoulders. The P designation indicates that polymer modified asphalt is specified.

4. **Microsurfacing.** Microsurfacing is a thin, polymer-modified asphalt emulsion mixture. Microsurfacing may be used to provide a new wearing course to arrest the oxidation of asphalt pavement, improve friction, or fill ruts. An existing pavement should not have excessive cracking or surface irregularities such as shoving. Cores should be taken to
determine the thickness and investigate if a stripping condition exists. Core data and life-cycle cost data should be reviewed with the Pavement Division for specific recommendations.

All pavement markings and raised pavement markers must be removed prior to placement of a microsurface. This should be included in the appropriate pavement-marking-removal pay items.

If a pavement cross section has irregularities that will require a leveling course, or ruts greater than 0.25 in. that will require a rut fill course, a multiple course microsurface should be specified. The designer should typically specify a multiple course microsurface. A single course microsurface may be specified in unique situations, such as a nearly new road in excellent condition where the only purpose of the microsurface is to restore friction.

Guidelines for selecting a pavement section for microsurfacing are as follows:

a. AADT. Microsurface may be used without regard to traffic volume.

b. Pavement Distresses. A microsurface may be used on a road with low severity surface cracks. Cracks will typically reflect through the microsurface in a short time period. Cracks should be sealed prior to the application of microsurface. Cracks wider than ¼ in. may need to be routed prior to sealing.

c. Rutting. Microsurface may be used to correct rutting.

d. Roughness. The IRI should be 130 or less. The pavement should not have severe distresses indicative of a pavement nearing the end of its life. Microsurfacing will not significantly improve surface roughness.

e. Friction. A pavement with a low FN should be considered for microsurface treatment. A microsurface will restore surface friction.

f. Surface Aging. A microsurface may be used to stop future deterioration of an asphalt pavement due to age hardening, oxidation, or minor raveling.

5. **Ultrathin Bonded Wearing Course.** Ultrathin bonded wearing course (UBWC) is a gap-graded, ultrathin hot-mix asphalt mixture applied over a thick polymer-modified asphalt emulsion membrane. The emulsion membrane seals the existing surface and produces high binder content at the interface of the existing roadway surface. The gap-graded mix is
placed with the emulsion membrane in one pass. Core data and life cycle cost data should be reviewed with the Director of Pavements for specific recommendations.

All thermoplastic pavement markings and raised pavement markers are to be removed prior to placement of a UBWC. The removal quantities should be included in the appropriate pavement-marking-removal pay-items quantities.

The pay item for UBWC should specify the gradation size as 4.75 mm, 9.5 mm, or 12.5 mm. In most applications, the 9.5mm gradation should be specified.

Guidelines for selecting a pavement section for UBWC are as follows:

a. AADT. UBWC may be used without regard to traffic volume.

b. Pavement Distresses. A UBWC may be used on a road with low to moderate severity surface cracks. Cracks should be sealed prior to the application of a UBWC. Cracks wider than ¼ in. may require routing prior to sealing.

c. Rutting. UBWC does not significantly correct rutting and should not be specified where existing ruts are greater than 0.25 in.

d. Roughness. The IRI should be 140 or less. The pavement should not have severe distresses indicative of a pavement nearing the end of its life. UBWC will moderately improve surface roughness.

e. Friction. A pavement with a low FN may be considered for a surface treatment. A UBWC will restore surface friction.

f. Surface Aging. A UBWC may be used to stop future deterioration of an asphalt pavement due to age hardening, oxidation, or moderate raveling.

6. **HMA Inlay or Overlay.** A thin HMA inlay (4.75 mm), or milling and filling (up to 2 in.), consists of milling the existing surface and replacing it with a new asphalt surface to the original surface elevation. A thin HMA overlay (4.75 mm) consists of profile milling or scarifying the existing surface and overlaying it with a new asphalt surface. For PM, the surface condition may have minor defects but should not have significant potholes, depressed cracks, or major distresses. Criteria to be used in considering a thin HMA inlay or overlay are as follows:
a. AADT. An HMA inlay or overlay may be used without regard to traffic volume.

b. Pavement Distresses. An HMA inlay or overlay will correct low to moderate severity surface cracks that may be associated with surface corrugations or washboarding.

c. Rutting. An HMA inlay or overlay will correct rutting.

d. Roughness. The IRI must be 150 or less. An HMA inlay or overlay will significantly improve the surface roughness. The designer should evaluate the condition of the existing pavement and adjust the design life accordingly.

e. Friction. A pavement with a low FN may be considered for an HMA inlay or overlay surface treatment. An HMA inlay or overlay will restore surface friction.

f. Surface Aging. An HMA inlay or overlay may be used to replace an aged, oxidized, or raveled surface.

7. Hot In-Place Recycling (HIR) is the process of heating and softening the existing asphalt pavement for processing. HIR is limited in depth to less than 2 in. (50 mm). After heating, the asphalt material is picked up and remixed with admixtures, spread back onto the surface of the roadway, and then compacted, all in one operation. Pavements with structural distresses are not good candidates for HIR. The expected service lives of the various HIR rehabilitation techniques, when undertaking a life-cycle economic analysis, generally fall within the following ranges:

- HIR with surface treatment: 4 - 6 years
- HIR with HMA overlay: 7 - 10 years

Criteria to be used in considering a thin HMA inlay or overlay are as follows:

a. AADT. HIR may be used without regard to traffic volume.

b. Pavement Distresses. HIR will address oxidation (aging) and most surface related distresses, i.e., cracking confined to the surface of the pavement.

c. Rutting. HIR will correct surface rutting.
d. Roughness. The IRI must be 150 or less. HIR will significantly improve the
surface roughness. The designer should evaluate the condition of the existing
pavement and adjust the design life accordingly.

e. Friction. A pavement with a low FN may be considered for a HIR surface
treatment. HIR will restore surface friction.

f. Surface Aging. HIR may be used to replace an aged, oxidized, or raveled surface.

8. Cold recycling (CR) reuses the existing asphalt pavement by milling to a depth of 3 to 4
in. (75-100 mm), mixing the millings with a recycling agent (asphalt emulsion), and paving
and compacting the cold-recycled mix. CR has been successfully used on pavements with
a higher degree of cracking that would normally required removal of the cracked surface
and a thick overlay. Instead, the top portion of the existing pavement is recycled, cracks
are discontinued and a thin overlay is usually applied over the cold recycled asphalt
pavement. Cold recycling which includes both Cold In-Place Recycling (CIR) and Cold
Central Plant Recycling (CCPR) is applicable for urban or rural roadways with high or low
volumes of traffic. The CIR process calls for milling the existing pavement, mixing various
recycling agents into the mixture, and then spreading the material across the pavement
width for compacting. The CCPR process is the same except the material is transported to
a central plant location for mixing and then is transported back to the site for placement
and compaction.

For CR projects, an existing roadway assessment, structural capacity assessment, materials
properties assessment, geometric assessment of the existing and proposed sections,
constructability assessment, and an economic assessment must be conducted.

The expected service lives of various CR rehabilitation techniques, when undertaking a
life-cycle economic analysis, generally fall within the following ranges:

- CIR with surface treatment . . . . . . . 6 - 10 years
- CIR with HMA overlay . . . . . . . . . . 7 - 20 years
- CCPR with surface treatment . . . . . . . 6 - 10 years
- CCPR with HMA overlay . . . . . . . . . . 12 - 20 years
Criteria to be used in considering a thin HMA inlay or overlay are as follows:

a. **AADT.** CR may be used without regard to traffic volume; however, maintenance of traffic (MOT) will have to be considered. A traffic assessment should be performed.

b. **Pavement Distresses.** CR can rehabilitate cracked pavements which are structurally sound and have well-drained bases. The CR process destroys existing crack patterns and produces a crack free layer for the new surface course such as an HMA or an asphalt surface treatment. For CR to be effective in mitigating cracking, as much of the existing asphalt pavement layer should be treated as possible. Typically, at least 70 percent of the existing asphalt pavement thickness needs to be treated in order to mitigate the reflection cracking. Treatment depth is also affected by the maximum depth that can be treated at one time.

c. **Rutting.** CR will correct surface rutting.

d. **Roughness.** The IRI must be 150 or less. CR will significantly improve the surface roughness. The designer should evaluate the condition of the existing pavement and adjust the design life accordingly.

e. **Friction.** A pavement with a low FN may be considered for CR and surface treatment. CR with an overlay will restore surface friction.

f. **Surface Aging.** CR with an overlay may be used to replace an aged, oxidized, or raveled surface.

### 304-19.02 PCCP PM Treatments

1. **Crack Sealing.** Crack sealing consists of the cleaning and sealing of open cracks or joints in PCCP to minimize the entry of moisture and debris. Cracks must be clean and dry and may be routed prior to sealing. This PM treatment may be periodic once more cracks develop as the pavement ages.

2. **PCCP Sawing and Sealing Joints.** Contraction joints and longitudinal joints should be inspected periodically and cleaned and resealed as required. For PM, timely sealing of the joints minimize dirt and moisture from entering the joints. Rigid pavement, 8 to 10 years old, should be inspected. If, on inspection, 10% of the joints have loose, missing, or
depressed sealant, sawing and sealing of the joints should be considered. The joints should be sawed to remove old sealant and to reshape the joint-seal reservoir.

3. **Retrofit Load Transfer.** This consists of retrofitting of dowels in jointed PCCP to re-establish load transfer across random cracks. The pavement performance is improved by keeping the elevation of adjacent panels at the same elevation and stops increases in the IRI due to faulting. This work consists of the cutting of slots, placing new dowels or reinforcing bars, then cementing them into place. The pavement may be profiled to improve smoothness after the retrofit load transfer is complete.

4. **Surface Profiling.** This is a procedure used to restore or improve pavement rideability by removing surface defects that develop from traffic loading and environmental conditions. Surface profiling enhances surface friction of an existing pavement surface. The resulting corduroy-like surface provides ample channels for water to escape the surface. Surface profiling is recommended to restore rideability if faulting causes the IRI to exceed 150. A faulted pavement must be repaired with retrofit load transfer prior to surface profiling.

5. **Partial-Depth Patching.** This is primarily used to improve ride quality. It should be limited to the upper one third of the concrete-pavement depth. The area to be patched should be sawed, and all unsound material removed prior to placement of patch material.

6. **Full-Depth Patching.** This consists of complete removal of a deteriorated section of concrete pavement for a full lane width and patching it with new concrete. Full-depth patching may be used to restore pavement rideability and to replace deteriorated joints and cracks. Full-depth patching details are shown on the INDOT Standard Drawings. Isolated cracked D-joints that have spalled out may be patched; however, patching would be considered a short term fix since the remainder of the joints will soon become distressed. If a pavement is D-joint cracked, a slab-reduction technique and overlay should be used.

7. **Underseal.** This consists of pumping flowable asphalt or cement material into voids under concrete pavement. This will stabilize the slab and minimize rocking and pumping, and extend the life of the pavement. Pavements with open-graded subbase should not be undersealed. Falling weight deflectometer (FWD) testing must be done in advance of undersealing to determine locations and material quantities.

8. **Slab Jacking.** This consists of raising a settled slab to its original profile grade by pumping flowable material underneath. This technique may be used on one or several panels to restore rideability. Panels should be intact with no mid-panel cracking.
9. **Stitching.** This treatment involves drilling and inserting reinforcing steel at approximately 45-deg angles across longitudinal cracks and joints in accordance with the specifications. This technique is used to prevent longitudinal cracks or joints from faulting.

### 304-20.0 LIFE-CYCLE COST ANALYSIS (LCCA)

Life-Cycle Cost Analysis is an economic evaluation technique that builds on the principles of economic analysis to evaluate the overall long-term solutions for each type of project. LCCA considers initial and future agency, user, and other relevant costs over the life of alternatives discounted to provide comparative costs. This technique allows a project’s cost to be compared over a specified time period. The selection of design alternatives should be made based on an LCCA sensitivity analysis for pavement life costs.

This section provides the methodology to perform an LCCA for a pavement project. Resources are available for further explanation of LCCA, such as FHWA SA-98-079, *Life-Cycle Cost Analysis in Pavement Designs*.

### 304-20.01 General Requirements

An LCCA will be completed on each alternative for a New Alignment, Reconstruction, or Rehabilitation (Structural) project. In the simplest situation, an LCCA evaluates costs associated with two or more particular strategies or design scenarios over an analysis period including the initial construction and at least one succeeding rehabilitation activity. These costs for the alternate scenarios or money flows are discounted to the present time. A comparison of the net present value of the scenarios is made to provide information regarding one of the factors involved in the selection of a pavement design.

The economic evaluation of two feasible design strategies or design scenarios has no relation to the method of financing, or the total cost of the project. Inflation is not a factor in the evaluation since two or more scenarios’ cash flows are being compared over the same time period with presumably the same inflation effects. Constant real dollars should be used in the LCCA, and then the budget analysis should decide funding sources, inflation rate, and cash-flow requirements.

An LCCA will be required for a New Alignment, Reconstruction, or Rehabilitation (Structural) project with mainline pavement of more than 10,000 yd² for determination of pavement type. A LCCA should be completed for all projects where costs of different equitable treatments are close (≤ 10% difference). A least cost of ownership analysis, (cost analysis = $/lane/mile/year) is also
required for each treatment identified in Section 304-19.0 to compare preventive maintenance preservation treatments with differing design lives. See Figure 304-14A.

Two scenarios being evaluated with a total net present value within 10% difference (15% for a preservation project with an initial cost as calculated for a cost analysis of less than $750,000) are considered to be essentially the same. Other factors should be used to make the final selection such as initial costs, constructability, work-zone traffic control, and user delay costs.

304-20.02 Definitions

304-20.02(01) Analysis Period

The analysis period is the number of years over which the pavement-life-cycle analysis is conducted. The analysis period should include the initial pavement cost and the cost of at least one subsequent rehabilitation. The analysis period should be at least 50 years in comparing new pavements. In comparing treatments with lesser design lives the number of years may be less.

304-20.02(02) Discount Rate

The discount rate is used to equate the cash flows to present worth and determine Equivalent Uniform Annual Cost (EUAC). For general purposes, a 4% discount rate can be assumed. However, a range of rates from 0% to 10% can be used to determine if the alternate scenarios are discount-rate sensitive. The results of the sensitivity analysis should be shown. See FHWA SA-98-079.

304-20.02(03) Equivalent Uniform Annual Cost (EUAC)

The EUAC is the combining of initial capital costs and all future expenses into equal annual payments over the analysis period.

\[ EUAC = (PW) \left[ \frac{i(1+i)^n}{(1+i)^n - 1} \right] \]

Where: \(PW\) = present worth
\(i\) = discount rate
\(n\) = number of years from year zero.
LCCA design life is the estimated service life of the pavement. The design life shown in Figure 304-14A, Pavement Design Life, should be used for the initial, maintenance, or rehabilitation option.

The design life of the pavement should be varied to test the LCCA for sensitivity based on the existing pavement condition, past performance, or the condition of the drainage system. The design life used for the sensitivity analysis should be documented.

The Office of Pavement Engineering will maintain a listing of historical bid summaries associated with pavement construction, rehabilitation, and maintenance contract costs identified as part of the proposed LCCA. The pavement designer should utilize these costs to compare life-cycle costs of different pavement treatments.

A factor in identifying and performing economic analyses of alternatives in the design of new pavement construction or the repair and rehabilitation of existing pavement is the life cycle of the alternative under consideration. The life-cycle cost includes the initial capital cost of construction, future maintenance, and future rehabilitation cost estimates. The life-cycle cost may also include user-delay costs during construction and rehabilitation, user vehicle operating and accident expenses, engineering fees, or other expenditures over the life of the pavement. It may also include the residual value, or salvage value of the pavement, at the end of the analysis time period. The pavement designer should use the service lives as shown in Figure 304-14A for the rehabilitation alternatives over the life of the pavement.

The cost of work-zone traffic control and the cost of user delays during construction between designs may have a significant effect on the analysis. These costs should be quantified for the designs. LCCA costs can include the following:
1. **Agency Costs.** These include the following:

   a. initial construction costs;
   b. future construction or rehabilitation costs, e.g., overlays, seal coats, or reconstruction;
   c. maintenance costs which recur throughout the design period;
   d. salvage return or residual value at the end of the design period, which may be a negative cost;
   e. engineering and administration costs; and
   f. traffic-control costs if they are involved.

2. **User Costs.** These include the following:

   a. travel time;
   b. vehicle operation;
   c. accidents;
   d. discomfort; and
   e. time delay and extra vehicle operating costs during resurfacing or major maintenance

304-20.02(06) **Present Worth (PW)**

The PW is the value of money at year zero of future expenditures. The future cash flow is discounted by the discount rate to determine PW. The equation for the present worth of a future overlay is as follows:

\[
PW = \frac{1}{(1 + i)^n} \left( F \right)
\]

Where:

- \( F = \) future construction cost
- \( i = \) discount rate
- \( n = \) number of years from year zero.

Note: INDOT uses probabilistic LCCA and present value for pavement alternative analysis, not EUCA.
**304-20.02(07) Salvage Value (SV)**

Salvage value is the residual value of the pavement’s remaining service life at the end of the analysis period. As an example, if the pavement surface has 5 yr of remaining life at the end of the analysis, the pavement has a remaining value which has not been used. SV is defined as the construction cost of the last cycle times the ratio of the remaining service years to the last cycle design life. The SV of the pavement is calculated from the equation as follows:

\[ SV = C \left( \frac{RL}{DL} \right) \]

Where:  
- \( C \) = last cycle construction cost, $  
- \( RL \) = remaining service life, yr  
- \( DL \) = last cycle design life, yr

**304-20.03 Analysis Steps**

An example of LCCA strategy follows. Differing pavement work types with different design lives may be used in alternate LCCA strategies:

**304-20.03(01) HMA Pavement**

50 year analysis period  
20 year design life

1. **Preventive Maintenance Treatment:**  
   - Joint/Crack Seal, at Year 3, 25% Longitudinal Joint Seal  
   - Joint/Crack Seal, at Year 6, 50% Longitudinal Joint Seal  
   - Joint/Crack Seal, at Year 9, 75% Longitudinal Joint Seal  
   - Joint/Crack Seal, at Year 12, 15, and 18, 100% Longitudinal Joint Seal

2. **Rehabilitation:**  
   - Mill 1” and two-layer Overlay 4” =1½” Surface on 2½” Intermediate, at Year 20  
   - Note: Variable Depth Aggregate Wedge Shoulder maybe necessary at Year 20.
3. Preventive Maintenance Treatment:
   Joint/Crack Seal, at Year 23, 25% Longitudinal Joint Seal
   Joint/Crack Seal, at Year 26, 50% Longitudinal Joint Seal
   Joint/Crack Seal, at Year 29, 75% Longitudinal Joint Seal
   Joint/Crack Seal, at Year 32, 100% Longitudinal Joint Seal

4. Preventive Maintenance Treatment:
   Mill 1” and Overlay 1½” Surface, with 1% full-depth HMA patching based on project area, at Year 35

5. Preventive Maintenance Treatment:
   Joint/Crack Seal, at Year 38, 25% Longitudinal Joint Seal
   Joint/Crack Seal, at Year 41, 50% Longitudinal Joint Seal

6. Preventive Maintenance Treatment:
   Mill 1” and Overlay 1½” Surface, with 1% full-depth HMA patching based on project area, at Year 44

7. Preventive Maintenance Treatment:
   Joint/Crack Seal, at Year 47, 25% Longitudinal Joint Seal

End LCCA, Salvage Value at Year 50 = $0.00 (3 years of Mill and Fill treatment remain)

304-20.03(02) PCCP

50 year analysis period
30 year design life

1. Preventive Maintenance Treatment:
   Transverse and Longitudinal Joint Seal, at Years 8, 16, and 24

2. Rehabilitation:
   Milling, scarification/profile and HMA two layer Overlay 4” =1½” Surface on 2½” Intermediate, with 3% patching based on D-1 Joint quantity, at Year 30

3. Preventive Maintenance Treatment:
   Joint/Crack Seal, at Year 33, 25% Longitudinal Joint Seal
   Joint/Crack Seal, at Year 36, 50% Longitudinal Joint Seal
Joint/ Crack Seal, at Year 39, 75% Longitudinal Joint Seal

4. Preventive Maintenance Treatment:
   Mill 1” and Overlay 1½” Surface, with 1% full-depth composite patching and with
   3% partial-depth HMA patching based on D-1 Joint quantity, at Year 42

5. Preventive Maintenance Treatment:
   Joint/ Crack Seal, at Year 45, 25% Longitudinal Joint Seal
   Joint/ Crack Seal, at Year 48, 50% Longitudinal Joint Seal

End LCCA, Salvage Value at Year 50 = $0.00

An LCCA example and major projects’ PCCP and HMA unit prices are available on the Pavement Engineering section of Standards and Specifications webpage, http://www.in.gov/dot/div/contracts/standards/.

304-21.0 TYPICAL PAVEMENT SECTIONS

304-21.01 HMA Pavement

Typical HMA mainline pavement sections are shown in Figures 304-21A through 304-21G.

304-21.02 PCC Pavement

Typical PCC mainline pavement sections and joint locations are shown in Figures 304-21P through 304-21R.

304-21.03 Miscellaneous Pavement Sections and Details

Underdrain details are shown in Figures 304-21I, 304-21J, 304-21K, 304-21O, 304-21T, 304-21U, 304-21V, and 304-21Y.

Ramp sections are shown in Figures 304-21H and 304-21S.

Concrete-curb sections are shown in Figures 304-21K, 304-21L, 304-21M, 304-21N, 304-21O, 304-21R, 304-21V, and 304-21Z.
PCCP Longitudinal Joint options on median shoulder sections are shown in Figure 304-21W.

Safety Edge sections are shown in Figure 304-21X.

Aggregate pavement section is shown in Figure 304-21AA.

Parking lot sections are shown in Figure 304-21BB.

Patching sections are shown in Figure 304-21CC, 304-21DD, 304-21EE, and 304-21FF.

### 304-22.0 PAVEMENT DESIGN REQUEST AND INSTRUCTIONS

A Pavement Design Request should be submitted to the Pavement Design Coordinator. An editable version of the Pavement Design Request and instructions are available for download from the Department’s website at [www.in.gov/dot/div/contracts/design/dmforms/](http://www.in.gov/dot/div/contracts/design/dmforms/). An incomplete Pavement Design Request will be returned without review.
<table>
<thead>
<tr>
<th>Pavement-Work Type</th>
<th>Design Life, Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCCP</td>
<td>30</td>
</tr>
<tr>
<td>PCCP over Existing Pavement</td>
<td>25</td>
</tr>
<tr>
<td>HMA Pavement with SMA</td>
<td>20</td>
</tr>
<tr>
<td>HMA with SMA Surface Overlay on Rubblized PCCP</td>
<td>20</td>
</tr>
<tr>
<td>HMA Pavement</td>
<td>20</td>
</tr>
<tr>
<td>HMA Overlay on CRCP</td>
<td>20</td>
</tr>
<tr>
<td>HMA Overlay on Rubblized PCCP</td>
<td>20</td>
</tr>
<tr>
<td>HMA Overlay on Cracked and Seated PCCP</td>
<td>12</td>
</tr>
<tr>
<td>HMA Overlay over Asphalt</td>
<td></td>
</tr>
<tr>
<td>Rehabilitation (≥ 3 layers)</td>
<td>18</td>
</tr>
<tr>
<td>Rehabilitation (2 layers)</td>
<td>15</td>
</tr>
<tr>
<td>Preventative Maintenance (1 layer)</td>
<td>9</td>
</tr>
<tr>
<td>HMA Overlay over PCCP</td>
<td></td>
</tr>
<tr>
<td>Rehabilitation (≥ 3 layers)</td>
<td>15</td>
</tr>
<tr>
<td>Rehabilitation (2 layers)</td>
<td>12</td>
</tr>
<tr>
<td>PCCP Joint Sealing</td>
<td>8</td>
</tr>
<tr>
<td>Ultrathin Bonded Wearing Course (UBWC)</td>
<td>9</td>
</tr>
<tr>
<td>Microsurface Overlay</td>
<td>8</td>
</tr>
<tr>
<td>Thin HMA Overlay with Profile Milling</td>
<td>9</td>
</tr>
<tr>
<td>Concrete Pavement Rehabilitation (CPR) Techniques</td>
<td>6</td>
</tr>
<tr>
<td>Chip Seal</td>
<td>4</td>
</tr>
<tr>
<td>Asphalt Crack Sealing, Rout and Seal</td>
<td>3</td>
</tr>
<tr>
<td>Asphalt Crack Filling</td>
<td>1</td>
</tr>
</tbody>
</table>

1 The performance period should be decreased to 8 yr for existing composite HMA over PCCP.

**PAVEMENT DESIGN LIFE**

*Figure 304-14A*
<table>
<thead>
<tr>
<th>Performance Criteria</th>
<th>Performance Limit at End of Design Life</th>
<th>Reliability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terminal IRI (in./mi)</td>
<td>Freeway: 160, Arterial, Urban: 190, Arterial, Rural: 200, Collector, Urban: 190, Collector, Rural: 200, Local: 200</td>
<td>90%, 80%, 85%, 90%, 75%, 70%</td>
</tr>
<tr>
<td>AC Bottom-Up Cracking, Alligator Cracking (% lane area)</td>
<td>Freeway: 10, Arterial, Urban: 20, Arterial, Rural: 25, Collector, Urban: 30, Collector, Rural: 35, Local: 35</td>
<td>90%, 85%, 80%, 75%, 70%</td>
</tr>
<tr>
<td>Permanent Deformation – AC only Pavement (in.)</td>
<td>Freeway: 0.40, Arterial, Urban: 0.40, Arterial, Rural: 0.40, Collector, Urban: 0.40, Collector, Rural: 0.40, Local: 0.40</td>
<td>90%, 85%, 80%, 75%, 70%</td>
</tr>
<tr>
<td>AC Thermal Fracture (ft/mi/lane)</td>
<td>Freeway: 500, Arterial, Urban: 500, Arterial, Rural: 500, Collector, Urban: 500, Collector, Rural: 500, Local: 500</td>
<td>90%, 85%, 80%, 75%, 70%</td>
</tr>
</tbody>
</table>

**PERFORMANCE CRITERIA FOR NEW OR REHABILITATION HMA PAVEMENT**

*Figure 304-14B*
<table>
<thead>
<tr>
<th>Performance Criteria</th>
<th>Performance limit at End of Design Life</th>
<th>Reliability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terminal IRI (in./mi)</td>
<td>Freeway: 160</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Urban: 190</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Rural: 200</td>
<td>85%</td>
</tr>
<tr>
<td></td>
<td>Collector, Urban: 190</td>
<td>80%</td>
</tr>
<tr>
<td></td>
<td>Collector, Rural: 200</td>
<td>75%</td>
</tr>
<tr>
<td></td>
<td>Local: 200</td>
<td>70%</td>
</tr>
<tr>
<td>Transverse Slab Cracking (%)</td>
<td>Freeway: 10</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Urban: 10</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Rural: 10</td>
<td>85%</td>
</tr>
<tr>
<td></td>
<td>Collector, Urban: 10</td>
<td>80%</td>
</tr>
<tr>
<td></td>
<td>Collector, Rural: 10</td>
<td>75%</td>
</tr>
<tr>
<td></td>
<td>Local: 10</td>
<td>70%</td>
</tr>
<tr>
<td>Mean Joint Faulting (in.)</td>
<td>Freeway: 0.15</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Urban: 0.20</td>
<td>90%</td>
</tr>
<tr>
<td></td>
<td>Arterial, Rural: 0.22</td>
<td>85%</td>
</tr>
<tr>
<td></td>
<td>Collector, Urban: 0.25</td>
<td>80%</td>
</tr>
<tr>
<td></td>
<td>Collector, Rural: 0.25</td>
<td>75%</td>
</tr>
<tr>
<td></td>
<td>Local: 0.25</td>
<td>70%</td>
</tr>
</tbody>
</table>

**PERFORMANCE CRITERIA FOR NEW OR REHABILITATION CONCRETE PAVEMENT**

*Figure 304-14C*
Asphalt General Input

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference Temperature, °F</td>
<td>70</td>
</tr>
<tr>
<td>Thermal Conductivity, Asphalt, BTU/h-ft-°F</td>
<td>0.63</td>
</tr>
<tr>
<td>Heat Capacity, Asphalt, BTU/lb-°F</td>
<td>0.31</td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td>0.35</td>
</tr>
</tbody>
</table>

### Volumetric Properties as Built

<table>
<thead>
<tr>
<th>Volumetric Properties as Built</th>
<th>NMAS, mm</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Binder Content, %</td>
<td>25.0</td>
<td>8.7</td>
</tr>
<tr>
<td></td>
<td>19.0</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>10.7</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>11.6</td>
</tr>
<tr>
<td>SMA 9.5</td>
<td></td>
<td>13.4</td>
</tr>
<tr>
<td>Air Voids, %</td>
<td>25.0</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>19.0</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>12.5</td>
<td>8</td>
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<tr>
<td></td>
<td>9.5</td>
<td>8</td>
</tr>
<tr>
<td>SMA 9.5</td>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Total Unit Weight, lb/ft³</td>
<td>25.0</td>
<td>144.4</td>
</tr>
<tr>
<td></td>
<td>19.0</td>
<td>143.8</td>
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<td></td>
<td>12.5</td>
<td>143.08</td>
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<td></td>
<td>9.5</td>
<td>142.6</td>
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<tr>
<td>SMA 9.5</td>
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MEPDG GENERAL INPUT VALUES
FOR ASPHALT PAVEMENT

Figure 304-14D
<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>QC/QA-HMA Category*</th>
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</thead>
<tbody>
<tr>
<td>AADTT &lt; 51</td>
<td>&lt; 0.3</td>
<td>2</td>
</tr>
<tr>
<td>51 ≤ AADTT &lt; 510</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>2</td>
</tr>
<tr>
<td>510 ≤ AADTT &lt; 1700</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>3</td>
</tr>
<tr>
<td>1700 ≤ AADTT &lt; 5100</td>
<td>10 ≤ ESAL &lt; 30</td>
<td>4</td>
</tr>
<tr>
<td>AADTT ≥ 5100</td>
<td>≥ 30</td>
<td>4</td>
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</table>

2-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>QC/QA-HMA Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 57</td>
<td>&lt; 0.3</td>
<td>2</td>
</tr>
<tr>
<td>57 ≤ AADTT &lt; 570</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>2</td>
</tr>
<tr>
<td>570 ≤ AADTT &lt; 1900</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>3</td>
</tr>
<tr>
<td>1900 ≤ AADTT &lt; 5700</td>
<td>10 ≤ ESAL &lt; 30</td>
<td>4</td>
</tr>
<tr>
<td>AADTT ≥ 5700</td>
<td>≥ 30</td>
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</table>

4-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>QC/QA-HMA Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 87</td>
<td>&lt; 0.3</td>
<td>2</td>
</tr>
<tr>
<td>87 ≤ AADTT &lt; 870</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>2</td>
</tr>
<tr>
<td>870 ≤ AADTT &lt; 2900</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>3</td>
</tr>
<tr>
<td>2900 ≤ AADTT &lt; 8700</td>
<td>10 ≤ ESAL &lt; 30</td>
<td>4</td>
</tr>
<tr>
<td>AADTT ≥ 8700</td>
<td>≥ 30</td>
<td>4</td>
</tr>
</tbody>
</table>

6-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>QC/QA-HMA Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 114</td>
<td>&lt; 0.3</td>
<td>2</td>
</tr>
<tr>
<td>114 ≤ AADTT &lt; 1140</td>
<td>0.3 ≤ ESAL &lt; 3</td>
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<td>1140 ≤ AADTT &lt; 3800</td>
<td>3 ≤ ESAL &lt; 10</td>
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<tr>
<td>3800 ≤ AADTT &lt; 11400</td>
<td>10 ≤ ESAL &lt; 30</td>
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<tr>
<td>AADTT ≥ 11400</td>
<td>≥ 30</td>
<td>4</td>
</tr>
</tbody>
</table>

8-LANE ROAD

* For open-graded mixtures OG 19.0 and OG 25.0, the QC/QA-HMA category is 4.

ESAL CATEGORY FOR QC/QA-HMA MIXTURES

Figure 304-15A
<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>HMA Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 51</td>
<td>&lt; 0.3</td>
<td>B</td>
</tr>
<tr>
<td>51 ≤ AADTT &lt; 510</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>B</td>
</tr>
<tr>
<td>510 ≤ AADTT &lt; 1700</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>C</td>
</tr>
<tr>
<td>AADTT ≥ 1700</td>
<td>≥ 10</td>
<td>D</td>
</tr>
</tbody>
</table>

2-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>HMA Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 57</td>
<td>&lt; 0.3</td>
<td>B</td>
</tr>
<tr>
<td>57 ≤ AADTT &lt; 570</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>B</td>
</tr>
<tr>
<td>570 ≤ AADTT &lt; 1900</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>C</td>
</tr>
<tr>
<td>AADTT ≥ 1900</td>
<td>≥ 10</td>
<td>D</td>
</tr>
</tbody>
</table>

4-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>HMA Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 87</td>
<td>&lt; 0.3</td>
<td>B</td>
</tr>
<tr>
<td>87 ≤ AADTT &lt; 870</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>B</td>
</tr>
<tr>
<td>870 ≤ AADTT &lt; 2900</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>C</td>
</tr>
<tr>
<td>AADTT ≥ 2900</td>
<td>≥ 10</td>
<td>D</td>
</tr>
</tbody>
</table>

6-LANE ROAD

<table>
<thead>
<tr>
<th>Initial AADTT, trucks per day</th>
<th>Design ESALs, millions</th>
<th>HMA Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADTT &lt; 114</td>
<td>&lt; 0.3</td>
<td>B</td>
</tr>
<tr>
<td>114 ≤ AADTT &lt; 1140</td>
<td>0.3 ≤ ESAL &lt; 3</td>
<td>B</td>
</tr>
<tr>
<td>1140 ≤ AADTT &lt; 3800</td>
<td>3 ≤ ESAL &lt; 10</td>
<td>C</td>
</tr>
<tr>
<td>AADTT ≥ 3800</td>
<td>≥ 10</td>
<td>D</td>
</tr>
</tbody>
</table>

8-LANE ROAD

MIXTURE TYPE FOR HMA MIXTURES

Figure 304-15B
<table>
<thead>
<tr>
<th>Outlet Protector Type</th>
<th>Outside</th>
<th>Median</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flattest</td>
<td>Steepest</td>
</tr>
<tr>
<td>1 (large)</td>
<td>10:1</td>
<td>2:1</td>
</tr>
<tr>
<td>2 (medium)</td>
<td>4:1</td>
<td>1:1</td>
</tr>
<tr>
<td>3 (small)</td>
<td>2:1</td>
<td>1:1</td>
</tr>
</tbody>
</table>

OUTLET PROTECTOR SLOPE LIMITS

Figure 304-18A
<table>
<thead>
<tr>
<th>Type 4 Pipe Used As Underdrain Pipe, ft</th>
<th>Video Inspection Pay Quantity, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,000 ≤ Type 4 Pipe &lt; 30,000</td>
<td>3,000</td>
</tr>
<tr>
<td>30,000 ≤ Type 4 Pipe &lt; 80,000</td>
<td>6,500</td>
</tr>
<tr>
<td>80,000 ≤ Type 4 Pipe &lt; 150,000</td>
<td>10,000</td>
</tr>
<tr>
<td>150,000 ≤ Type 4 Pipe &lt; 300,000</td>
<td>13,000</td>
</tr>
<tr>
<td>≥ 300,000</td>
<td>16,000</td>
</tr>
</tbody>
</table>

VIDEO INSPECTION CONTRACT QUANTITIES

Figure 304-18B
<table>
<thead>
<tr>
<th>Treatment</th>
<th>AADT</th>
<th>Pavement Distresses</th>
<th>Rutting, in.</th>
<th>IRI</th>
<th>Friction Treatment?</th>
<th>Surface Aging</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack Seal</td>
<td>Any</td>
<td>Low to Moderately Severe Transverse or Longitudinal Joints/Reflective Cracks</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Crack Fill</td>
<td>Any</td>
<td>Low to Moderately Severe Longitudinal Cold Joint, Reflective &amp; Edge Cracking Plus Low Severity Block Cracking</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Fog Seal</td>
<td>&lt; 5,000</td>
<td>Low-Severity Environmental Surface Cracks</td>
<td>n/a</td>
<td>n/a</td>
<td>No&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Seal Coat</td>
<td>&lt; 5,000</td>
<td>Low-Severity Environmental Surface Cracks</td>
<td>&lt; 0.25&lt;sup&gt;4&lt;/sup&gt;</td>
<td>n/a&lt;sup&gt;4&lt;/sup&gt;</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Microsurface</td>
<td>Any</td>
<td>Low-Severity Surface Cracks</td>
<td>Any</td>
<td>&lt; 130</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>UBWC</td>
<td>Any</td>
<td>Low to Moderately Severe Surface Cracks</td>
<td>&lt; 0.25&lt;sup&gt;4&lt;/sup&gt;</td>
<td>&lt; 140</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>HMA Inlay</td>
<td>Any</td>
<td>Low to Moderately Severe Surface Cracks</td>
<td>Any</td>
<td>&lt; 150</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Thin HMA Overlay w/Profile Milling</td>
<td>Less than 10 million, ESAL</td>
<td>Low to Moderately Severity Surface Cracks (For use on category 1,2 or 3 roads only)</td>
<td>&lt; 0.25&lt;sup&gt;4&lt;/sup&gt;</td>
<td>&lt; 150</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>HMA Overlay</td>
<td>Any</td>
<td>Low to Moderately Severe Surface Cracks</td>
<td>Any</td>
<td>&lt; 150</td>
<td>Yes</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. For mainline pavement.
2. Unless traffic can be adequately controlled.
3. Treatment may reduce skid numbers.
4. Treatment does not address this.

**HMA PREVENTIVE MAINTENANCE TREATMENTS**

*Figure 304-19A*
<table>
<thead>
<tr>
<th>Treatment</th>
<th>AADT(^1)</th>
<th>Pavement Distresses</th>
<th>Rutting</th>
<th>IRI</th>
<th>Friction Treatment?</th>
<th>Surface Aging</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack Seal</td>
<td>Any</td>
<td>Midpanel cracks with aggregate interlock</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Saw and Seal Joints</td>
<td>Any</td>
<td>&gt; 10% of joints with missing sealant, otherwise joints in good condition</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Retrofit Load Transfer</td>
<td>Any</td>
<td>Low to medium severity mid-panel cracks; pumping or faulting at joints &lt; 0.25 in.</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Surface Profiling</td>
<td>Any</td>
<td>Faulting &lt; 0.25 in.; poor ride; friction problems</td>
<td>n/a</td>
<td>n/a</td>
<td>Yes</td>
<td>n/a</td>
</tr>
<tr>
<td>Partial-Depth Patch</td>
<td>Any</td>
<td>Localized surface deterioration</td>
<td>n/a</td>
<td>n/a</td>
<td>Yes</td>
<td>n/a</td>
</tr>
<tr>
<td>Full-Depth Patch</td>
<td>Any</td>
<td>Deteriorated joints; faulting ≥ 0.25 in.; cracks</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Underseal</td>
<td>Any</td>
<td>Pumping; voids under pavement</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
<tr>
<td>Slab Jacking</td>
<td>Any</td>
<td>Settled slabs</td>
<td>n/a</td>
<td>n/a</td>
<td>No</td>
<td>n/a</td>
</tr>
</tbody>
</table>

Note:
\(^1\) On mainline pavement.

PCCP PREVENTIVE MAINTENANCE TREATMENTS

Figure 304-19B
NOTES:

1. 165 lb/yd² HMA Surface 9.5 mm
2. ___ lb/yd² HMA Intermediate
3. ___ lb/yd² HMA Base
4. ___ lb/yd² QC/QA-HMA Intermediate OG
5. ___ lb/yd² HMA Base
6. Subgrade Treatment, Type _____
7. Variable-Depth Compacted Aggregate, No. 53
8. Underdrain. See Figure 304-21 I for detail.
9. Safety edge as required for Surface and Intermediate layers. See Figure 304-21X for detail.
11. Liquid Asphalt Sealant required on Surface layer over longitudinal joint, 24" width.
12. Base seal is required under all open-graded HMA layers.
13. Configuration for median shoulder is the same as for an outside shoulder except width and slope.

* See Figure 304-21D for lay rate.

FULL-DEPTH HMA PAVEMENT WITH FULL-DEPTH SHOULDER WITH UNDERDRAIN

Figure 304-21A
NOTES:

1. 165 lb/yd^2 HMA Surface 9.5 mm
2. 330 lb/yd^2 HMA Intermediate 19.0 mm
3. 605 lb/yd^2 Min. HMA Base 25.0 mm
4. Subgrade Treatment, Type ______
5. Variable-Depth Compacted Aggregate, No. 53

6. Safety edge as required for Surface and Intermediate layers. See Figure 304-21X for detail.
7. Longitudinal joint adhesive required for Surface and Intermediate layers.
8. Liquid Asphalt Sealant required on Surface layer over longitudinal joint, 24" width.
9. Configuration for median shoulder is the same as for an outside shoulder except width and slope.

FULL-DEPTH HMA PAVEMENT WITH FULL-DEPTH SHOULDER
WITHOUT UNDERDRAIN

Figure 304-21B
FULL-DEPTH HMA PAVEMENT WITH HMA ON COMPACTED AGGREGATE SHOULDER WITH UNDERDRAIN

Figure 304-21C
<table>
<thead>
<tr>
<th>Full Depth HMA Thickness</th>
<th>Layer No.</th>
<th>Course</th>
<th>Lay Rate lb/yard²</th>
<th>Aggregate Size, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>275</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>13.0 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>275</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>385</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>13.5 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>385</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>14.0 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>440</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>14.5 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>495</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>15.0 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>550</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>15.5 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>605</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>16.0 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>660</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td>16.5 in.</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Base</td>
<td>715</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Intermediate OG</td>
<td>250</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Base</td>
<td>330</td>
<td>19.0</td>
</tr>
</tbody>
</table>
NOTES:
Mainline, Section with Shoulders
1. 165 lb/yd² HMA Surface 9.5 mm
2. 330 lb/yd² HMA Intermediate 19.0 mm
3. 605 lb/yd² min. HMA Base 25.0 mm
4. Subgrade Treatment, Type _____

Shoulders
5. 165 lb/yd² HMA Surface 9.5 mm
6. 330 lb/yd² HMA Intermediate 19.0 mm
5, 6, and 7 may be replaced by 10 in. equivalent thickness consisting of 4 in. Compacted Aggregate, No. 73 on 6 in. Compacted Aggregate, No. 53, Base.
7. 5.5 in. Compacted Aggregate, No. 53, Base.
    Depth equals Mainline HMA thickness minus 4.5 in.
8. Variable-Depth Compacted Aggregate, No. 53

9. Safety edge as required for Surface and Intermediate layers. See Figure 304-21X for detail.
11. Liquid Asphalt Sealant required on Surface layer over longitudinal joint, 24" width.

FULL-DEPTH HMA PAVEMENT WITH HMA ON COMPACTED AGGREGATE SHOULDER WITHOUT UNDERDRAIN

Figure 304-21E
NOTES:

Mainline, Section with Shoulders

1. ___ lb/yd² HMA Surface 9.5 mm
2. ___ lb/yd² HMA Intermediate 19.0 mm
3. ___ in. Compacted Aggregate, No. 53, Base
4. Subgrade Treatment, Type _____

Shoulders

5. ___ lb/yd² HMA Surface 9.5 mm
6. ___ lb/yd² HMA Intermediate 19.0 mm
5, 6, and 7 may be replaced by 10 in. equivalent thickness consisting of 4 in. Compacted Aggregate, No. 73 on 6 in. Compacted Aggregate, No. 53, Base.
7. ___ in. Compacted Aggregate, No. 53, Base.
Depth equal to 3.
8. Variable-Depth Compacted Aggregate, No. 53

9. Safety edge as required for Surface and Intermediate layers. See Figure 304-21X for detail.
11. Liquid Asphalt Sealant required on Surface layer over longitudinal joint, 24" width.

* See Figure 304-21G for lay rate.

HMA ON COMPACTED AGGREGATE PAVEMENT

Figure 304-21F
<table>
<thead>
<tr>
<th>HMA Pavement Thickness</th>
<th>Layer No.</th>
<th>Course</th>
<th>Lay Rate lb/yd²</th>
<th>Aggregate Size, mm</th>
<th>Layer Thickness in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0 inches</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>275</td>
<td>19.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>CA, No. 53, Base</td>
<td>-</td>
<td>-</td>
<td>6&quot;</td>
</tr>
<tr>
<td>4.5 inches</td>
<td>1</td>
<td>Surface</td>
<td>165</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>330</td>
<td>19.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>CA, No. 53, Base</td>
<td>-</td>
<td>-</td>
<td>5.5&quot;</td>
</tr>
<tr>
<td>4.5 inches</td>
<td>1</td>
<td>Surface</td>
<td>220</td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Intermediate</td>
<td>275</td>
<td>19.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>CA, No. 53, Base</td>
<td>-</td>
<td>-</td>
<td>5.5&quot;</td>
</tr>
<tr>
<td>5.0 inches</td>
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**TYPICAL HMA PAVEMENT ON COMPACTED AGGREGATE**

Figure 304-21G
Ramp:
1. 165 lb/yd² HMA Surface 9.5 mm
2. _____ lb/yd² HMA Intermediate
3. _____ lb/yd² HMA Base
4. _____ lb/yd² QC/QA-HMA Intermediate OG
5. _____ lb/yd² HMA Base
6. Subgrade Treatment, Type _____
7. Variable-Depth Compacted Aggregate, No. 53
8. Underdrain, See Figure 304-21 I for detail.

* See Figure 304-21D for lay rate.

NOTES:
9. Safety edge as required for Surface and Intermediate layers. See Figure 304-21X for detail.
11. Liquid Asphalt Sealant required on Surface layer over longitudinal joint, 24" width.
12. Base seal is required under all open-graded HMA layers.

FULL-DEPTH HMA RAMP

Figure 304-21H
NOTE:
Configuration for median shoulder is the same as for an outside shoulder.

UNDERDRAIN FOR HMA PAVEMENT
WITH FULL-DEPTH HMA SHOULDER

Figure 304-21 I
Aggregate for Underdrains
Pipe, Type 4, Circular, D = 6"

Top of Subgrade
D + 8"

Intermediate OG Layer

Slope Break Point

Edge of Travelway

2'-0"

Lane

Shoulder

Thickness

2'-0" + Pavement

Thickness

HMA Surface

HMA Intermediate

Compacted Aggregate, No. 53, Base

Shoulder

Interim O G Layer

Lane

HMA Surface

HMA Intermediate

Compacted Aggregate, No. 53, Base

HMA Surface

HMA Intermediate

Shoulder

Lane

Geotextile for Underdrains, Where Required

Aggregate for Underdrains

Pipe, Type 4, Circular, D = 6"

D + 8"

Top of Subgrade

6"
CONCRETE CURB AND GUTTER SECTION FOR HMA PAVEMENT WITH UNDERDRAIN

Figure 304-21K
CONCRETE CURB AND GUTTER SECTION FOR HMA PAVEMENT WITHOUT UNDERDRAIN

Figure 304-21L
CONCRETE CURB AND GUTTER SECTION FOR HMA OR PCCP PAVEMENT WITHOUT UNDERDRAIN

Figure 304-21M
MODIFIED CONCRETE CURB AND GUTTER SECTION FOR HMA OR PCCP PAVEMENT ON COMPACTED AGGREGATE WITHOUT UNDERDRAIN

Figure 304-21N
MODIFIED CONCRETE CURB AND GUTTER SECTION FOR HMA OR PCCP PAVEMENT WITH UNDERDRAIN

Figure 304-21 O
Mainline and Shoulders

1. PCCP
2. Subbase for PCCP (3 in. Coarse Aggregate No.8 on 6 in. Coarse Aggregate, No. 53)
3. Variable-Depth Compacted Aggregate, No. 53
4. Underdrain. See Figure 304-21T for detail.
5. Subgrade Treatment, Type _____
6. Longitudinal Joint or Longitudinal Construction Joint
7. Longitudinal Joint or Longitudinal Construction Joint, or no joint. See Figure 304-21W for detail.
8. Concrete Median Barrier
9. Safety edge as required. See Figure 304-21X for detail.

* Where underdrains are not required, Dense Graded Subbase should be used.

**Figure 304-21P**
NOTES:
1. PCCP
2. 165 lb/yd² HMA Surface 9.5 mm
   275 lb/yd² HMA Intermediate 19.0 mm
3. HMA Base 25.0 mm
4. Subbase for PCCP (3 in. Coarse Aggregate, No. 8 on 6 in.
   Coarse Aggregate, No. 53)
5. Variable-Depth Compacted Aggregate, No. 53
6. Underdrain. See Figure 304-21U for detail.
7. Subgrade Treatment, Type ______
8. Longitudinal Joint or Longitudinal Construction Joint
9. Longitudinal Joint or Longitudinal Construction Joint or
   no joint. See Figure 304-21W for detail.
10. For width < 8'-0", pavement type is per pavement
    design.
11. Safety edge as required. See Figure 304-21X for detail.

* Where underdrains are not required, Dense Graded Subbase
   should be used.

PCCP SECTION WITH HMA OUTSIDE SHOULDER

Figure 304-21Q
PCCP WITH CONCRETE CURB

Figure 304-21R
NOTES:
Ramp
① PCCP
② Subbase for PCCP (3 in. Coarse Aggregate, No. 8 on 6 in. Coarse Aggregate, No. 53)
③ Variable-Depth Compacted Aggregate, No. 53
④ Subgrade Treatment, Type ____
⑤ Underdrain. See Figure 304-21T for detail.
⑥ Longitudinal Joint or Longitudinal Construction Joint, 14-ft max. spacing between two Longitudinal Joints.
⑦ For multi-lane ramp, see Figure 304-21P.

PCCP RAMP

Figure 304-21S
Type ____, Subgrade Treatment, Pipe, Type 4, Circular, D = 6"
Aggregate For Underdrains

No. 53 Compacted Aggregate,
2'-0" D + 8"
1'-0"

Slope Break Point

PCCP

Subbase for PCCP

Aggregate, No. 8
3 in. Coarse
6 in. Coarse

Aggregate, No. 53

Figure 304-21T

UNDERDRAIN FOR PCCP

Subgrade Treatment,
Type ____

Geotextile for Underdrains Where Required

3 in. Coarse Aggregate, No. 8
6 in. Coarse Aggregate, No. 53

Subbase for PCCP

Pipe, Type 4, Circular, D = 6"

3 in. Coarse Aggregate, No. 8
6 in. Coarse Aggregate, No. 53
Type ____, Subgrade Treatment, Pipe, Type 4, Circular, D = 6”

No. 53 Compacted Aggregate, 2’-0” D + 8”

HMA Surface HMA Intermediate HMA Base

6” PCCP

Type Subgrade Treatment, Aggregate, No. 8 3 in. Coarse Aggregate, No. 53 6 in. Coarse

Subbase for PCCP

3 in. Coarse Aggregate, No. 8 6 in. Coarse Aggregate, No. 53

Subbase for PCCP

Subgrade Treatment, Type ....

Aggregate For Underdrains

PCCP

Geotextile for Underdrains per Geotechnical Report

EDGE OF TRAVEL LANE

Paved Shoulder Width

Usable Shoulder Width

36’-0"

6’

2’-0"

2’-0"

2’-0”

Figure 304-21U

UNDERDRAIN FOR PCCP WITH HMA SHOULDER
Pipe, Type 4, Circular, D = 6" Where Required

Geotextile for Underdrains, Underdrains Aggregate For (Req'd w/ Curb Installation)

Subgrade Treatment, PCCP Standard Drawing E 605-CCIN-01 for curb detail.

NOTE:

Standard Drawing E 605-CCIN-01 for curb detail.

UNDERDRAIN FOR CURBED PCCP

Figure 304-21V
MEDIAN EDGE OF CONCRETE PAVEMENT
LONGITUDINAL JOINT OPTIONS

Figure 304-21W
SAFETY EDGE

Figure 304-21X
RETROFIT UNDERDRAIN

Figure 304-21Y
HMA Pavement with Concrete Curb and No Underdrain

HMA Base. See Figure 304-21K for lay rate.

Figure 304-21Z
NOTES:

1. 4 in. Compacted Aggregate, No. 73
2. 6 in. Compacted Aggregate, No. 53, Base
3. Subgrade Treatment, Type ....
4. Variable-Depth Suitable Material

AGGREGATE PAVEMENT

Figure 304-21AA
HMA Section:

1) HMA on Compacted Aggregate Pavement (AADTT < 50)
   165 lb/yd² QC/QA, HMA, 1, 64, Surface 9.5 mm on
   275 lb/yd² QC/QA, HMA, 1, 64, Intermediate 19.0 mm on
   6 in. Compacted Aggregate, No. 53, Base, on
   Subgrade Treatment Type ____

2) HMA on Compacted Aggregate Pavement (AADTT < 250)
   165 lb/yd² QC/QA, HMA, 1, 64, Surface 9.5 mm on
   385 lb/yd² QC/QA, HMA, 1, 64, Intermediate 19.0 mm on
   5 in. Compacted Aggregate, No. 53, Base, on
   Subgrade Treatment Type ____

3) HMA on Compacted Aggregate Pavement (AADTT < 500)
   165 lb/yd² QC/QA, HMA, 1, 64, Surface 9.5 mm on
   495 lb/yd² QC/QA, HMA, 1, 64, Base 25.0 mm on
   4 in. Compacted Aggregate, No. 53, Base, on
   Subgrade Treatment Type ____

PCCP Section:

1) AADTT < 50
   7 in. of PCCP at 14-ft joint spacing with 1-in. dowel bar on
   6 in. of Dense Graded Subbase on
   Subgrade Treatment Type ____

2) AADTT < 500
   7.5 in. of PCCP at 15-ft joint spacing with 1-in. dowel bar on
   6 in. of Dense Graded Subbase on
   Subgrade Treatment Type ____

NOTE: These pavement sections (HMA or PCCP) should not be used for Rest Area Parking.

PARKING LOT PAVEMENT SECTIONS

Figure 304-21BB
PARTIAL-DEPTH HMA PATCH

CONCRETE PAVEMENT

COMPOSITE PAVEMENT

ASPHALT PAVEMENT

PARTIAL-DEPTH HMA PATCH

Figure 304-21CC
NOTES:
3. For any subsequent overlay mill, then overlay with HMA Surface course.

FULL-DEPTH CONCRETE PATCH IN COMPOSITE PAVEMENT

Figure 304-21DD
1. The assembly of #4 and #6 bars should be installed at half the depth of the existing subbase.

2. For any subsequent overlay mill, then overlay with HMA Surface course.

FULL-DEPTH COMPOSITE PATCH, INVERTED T

Figure 304-21EE
FOR CRACKED AND SEATED PAVEMENT OR CRCP

Full-Depth Concrete Patch

NOTE: The assembly of #4 and #6 bars should be installed at half the depth of the existing subbase.

Figure 304-21FF
NOTE: This chapter is currently being re-written and its content will be included in Chapter 305 in the future.

CHAPTER 46

Intersections At-Grade

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CHAPTER 46

INTERSECTIONS AT-GRADE

This Chapter discusses the geometric design of an at-grade intersection. The intersection is an important part of the highway system. The operational efficiency, capacity, safety, and cost of the system depend largely upon its design, especially in an urban area. The primary objective of intersection design is to reduce potential conflicts between vehicles, bicycles, and pedestrians while providing for the convenience, ease, and comfort of those traversing the intersection.

46-1.0 GENERAL DESIGN CONTROLS

46-1.01 Design Speed

The design speed for the intersection approaches should be equal to the design speed of the approach facility. However, this should not discourage the designer from using reduced geometric criteria if necessary to produce a more desirable result (e.g., to reduce driver speed prior to the intersection). If the designer can reasonably justify the design exception, the reduced criteria should be considered. To determine if reduced criteria may be applicable, the designer should consider the factors as follows:

1. heavy development along the road;
2. a posted speed limit lower than the design speed;
3. stopping sight distance;
4. adverse impacts to property owners;
5. adverse impacts to the environment;
6. low AADT;
7. construction costs;
8. adequate advance signing;
9. stop-sign control;
10. violation of driver expectancy;
11. T intersection;
12. short frontage road (See Section 45-7.04); or
13. access road with only one outlet (See Section 45-7.04).

For more information on using reduced criteria for a frontage road or local-road intersection, see Section 45-7.04.
46-1.02 Intersection Alignment

All legs of an intersection should be on a tangent section. Where a minor road intersects a major road on a horizontal curve, the geometric design of the intersection becomes significantly more complicated, particularly for sight distance, turning movements, channelization, and superelevation. If relocation of the intersection is not practical, the minor road may be realigned to intersect the major road perpendicular to the tangent at a point on the horizontal curve. Although an improvement, this arrangement may still result in difficult turning movements if the major road is superelevated. Intersection sight distance should be considered.

Roadways should intersect at right angles. An intersection at an acute angle is undesirable for the reasons as follows:

1. Vehicular turning movements become more restricted.

2. The accommodation of large trucks may require additional pavement and channelization.

3. The exposure time for vehicles and pedestrians crossing the main traffic flow is increased.

4. The driver’s line of sight for one of the sight triangles becomes restricted.

The angle of intersection should be within 20 deg of perpendicular. This amount of skew can often be tolerated because the impact on sight lines and turning movements may not be significant. Under restricted conditions where obtaining the right of way to straighten the angle of intersection would be impractical, an intersection angle up to 30 deg from perpendicular may be used. A more-acute intersection angle may warrant more positive traffic control (all-stop or traffic signalization) or geometric improvements (realignment, greater corner sight distance). The practice of realigning roads that intersect at an acute angle as shown in Figure 46-1A, Treatment for Skewed Intersection, diagrams A and B, has proven to be beneficial. The practice of constructing short-radius curves on the side-road approaches to achieve a right-angle intersection should be avoided where practical, because it may result in increased lane encroachments.

46-1.03 Intersection Profile

The designer should avoid combinations of grade lines that make vehicular control difficult at an intersection. The following criteria will apply.
46-1.03(01) Approach Grade

The grades of the intersecting highways should be as flat as practical on those portions that will be used for storage of stopped vehicles. This is referred to as the storage platform. The storage platform grade should be 0.5%, not to exceed 2% where practical, on each intersecting leg within the expected storage distance on the leg (see Section 46-4.02). At a minimum, the storage platform should be at least 50 ft long where there are less than 10% trucks, or 100 ft long where there are 10% or more trucks. A grade of steeper than 3% should be avoided, if practical. However, a grade through the intersection must reflect the practicalities of matching the basic profiles of the intersecting roadways and shoulders. The intersecting-roadway grade should not exceed the grade differences ($\Delta G$) as defined in Section 46-1.03(03) with respect to the mainline cross slope. The mainline shoulder or turn-lane cross slope and the mainline cross slope should not exceed the breakover cross slope differences shown in Figure 46-3F.

46-1.03(02) Cross-Section Transition

One or both of the approaching minor-road legs of an intersection may need to be transitioned (or warped) to meet the cross section of the major road. The designer should consider the following:

1. **Stop-Controlled.** Where the minor road is stop controlled, the profile and cross section of the major road will be maintained through the intersection. The cross slope of the stop-controlled leg will be transitioned to match the major-road cross slope and profile.

2. **Signalized Intersection.** At a signalized or a potentially signalized intersection, the cross section of the minor road will be transitioned to meet the profile and cross slope of the major road. If both intersecting roads have approximately equal importance, the designer may want to consider transitioning both roadways to form a plane section through the intersection. Where compromises are necessary between two such major roadways, the smoother riding characteristics should be provided for the roadway with the higher traffic volume and operating speed.

3. **Transition Rate.** Where one or both intersecting roadways are transitioned, the designer must determine the length and rate of transition from the normal section to the modified section. See Figure 46-1B, Pavement Transition Through Intersection. The transition should be designed to meet the general principles of superelevation transition which apply to that roadway (i.e., open-road or low-speed urban street condition). See Section 43-3.0 for a complete discussion on superelevation development. Where these criteria are applied to the transition rate, the applied design speed is typically one of the following:
a. 30 mph for a stop-controlled leg;  
b. the highway design speed for a free-flowing leg; or  
c. the highway design speed for each leg of a signalized intersection.

At a minimum, the approaching legs of an intersection should be transitioned within the curb or curve radius length of the intersection consistent with practical field conditions (see Figure 46-1B, Pavement Transition Through Intersection).

46-1.03(03) Vertical Profile

Where the cross section of the minor road is warped to meet that of the major road, this will result in angular breaks for traffic on the minor road if no vertical curve is inserted. If the vertical curve at the intersection cannot be designed for full stopping sight distance as discussed in Item 1 below, lighting of the intersection should be considered. The following options are provided in order from the most desirable to the least desirable (see Figure 46-1C, Vertical Profile of Intersecting Road).

1. Vertical Curve (SSD). A vertical curve should be used through an intersection which meets the criteria for stopping sight distance as described in Chapter Forty-four. For stop-controlled legs, the vertical curve should be designed to meet a design speed of 30 mph. At free-flowing legs or at a signalized intersection, the design speed of the roadway should be used to design the vertical curve.

2. Sag Vertical Curve (Comfort). For a sag vertical curve, the next most desirable option is to design the sag to meet the comfort criteria. The length of vertical curve can be determined as follows:

\[
L = \frac{AV^2}{46.5}
\]

Where:

- \( L \) = length of vertical curve, ft
- \( A \) = algebraic difference between grades, %
- \( V \) = design speed, mph

3. Vertical Curve (Minimum Comfort). Under restricted conditions where a design based on SSD or comfort is not practical and where the design speed is 30 mph or lower, a vertical curve at an intersection approach may be based on the formulas as follows:

\[
K = (0.10V)^2 \quad \text{(Sag Curve)}
\]
\[ K = (0.07V)^2 \quad \text{(Crest Curve)} \]

\[ L = KA \]

Where:

\[ K = \text{the horizontal distance in feet needed to produce a 1\% change in the gradient along the curve} \]
\[ A = \text{algebraic difference between the two tangent grades, \%} \]
\[ V = \text{design speed, mph} \]
\[ L = \text{length of vertical curve, ft} \]

4. **Angular Breaks.** Under restricted conditions, it may be impractical to provide vertical curves on the approaches, as angular breaks are necessary through the intersection. Angular breaks may allow other intersection geometric features, such as sight distance, storage platform, and drainage, to function better. Figure 46-1C, Vertical Profile of Intersecting Road, provides a schematic of vertical profiles through an intersection. The figure also indicates maximum angular breaks for a design speed of 30 mph or lower. For a higher design speed, a vertical curve as discussed in Items 1 and 2 above should be used. Where angular breaks are used, the minimum chord distance between angle points should be at least 15 ft.

46-1.03(04) **Drainage**

The profile and transitions at each intersection should be evaluated for impacts on drainage. This may require spot elevations to be shown for an intersection which may have exceptional drainage problems (e.g., an intersection which occurs in a sag vertical curve).

46-1.04 **Capacity and Level of Service**

The Office of Environmental Services will perform a capacity analysis of the intersection during the preparation of the Engineer’s Report. This analysis will influence geometric design features including the number of approach lanes, lane widths, channelization, and number of departure lanes. These determinations will be based on a selected level of service and design-year traffic (i.e., 20 years into the future). Level-of-service criteria are shown in the geometric-design figures in Chapters Fifty-three through Fifty-six. Once the level of service and design traffic volume are determined, the detailed capacity analysis is performed using the *Highway Capacity Manual* and the criteria provided in Chapter Forty-one.
46-1.05  Types of Intersections

46-1.05(01)  Number of Legs

An at-grade intersection is usually a 3-leg (T or Y shape), 4-leg, or multi-leg design. An individual intersection may vary in size and shape and may be non-channelized, flared, or channelized. The principal factors which affect the selection of intersection type and its design characteristics are the design hourly traffic volume, turning movements, traffic character or composition, design speed, intersection angle, topography, desired type of operations, and safety.

A multi-leg intersection is that with five or more intersection legs, and should be avoided where practical. Where traffic volume is light and stop control is used, it may be satisfactory to have all legs intersect at a common paved area. At other than a minor intersection, safety and efficiency are improved by rearrangement that removes some conflicting movements from the major intersection. This may be accomplished by realigning one or more of the intersecting legs and combining some of the traffic movements at adjacent subsidiary intersections or, making one or more legs one-way away from the intersection.

46-1.05(02)  Public-Road Approach

The warrants for each type of public-road approach are as follows:

1. Public-Road Approach Type A. This approach should be used where the mainline shoulder is unpaved, or, if paved, is less than 8 ft in paved width.

2. Public-Road Approach Type B. This approach should be used where the mainline shoulder is paved, and is 8 ft or wider in paved width. A paved shoulder of this width or greater will encourage use by a right-turning vehicle to clear the mainline traffic lane when decelerating for the turn.

Public-road approach types A and B are designed to accommodate a design vehicle of WB-50 or smaller, which makes a right-hand turn beginning and ending in the traffic lanes. Right-turn lanes are not provided for these approaches. Either of these approaches should be used for a public road serving a residential, light-commercial, or light-industrial area.

3. Public-Road Approach Type C. This approach should be used where the mainline shoulder is paved, is 8 ft or wider in paved width, and an auxiliary right-turn lane along the mainline is warranted due to the right-turning traffic volume. This approach is designed to
accommodate a design vehicle of WB-50 or smaller without encroaching onto the adjoining traffic lane. It will also accommodate a WB-65 design vehicle if a portion of the adjoining traffic lane is utilized. This approach should be used for a public road serving a residential, light-commercial, or light-industrial area.

4. **Public Road Approach Type D.** This approach should be used where the mainline shoulder is paved, is 8 ft or wider in paved width, and an auxiliary right-turn lane along the mainline is warranted due to the right-turning traffic volume. This approach is designed to accommodate a design vehicle of WB-65 or smaller. This approach should be used where two Department-maintained routes intersect, or for a public road serving a commercial area, heavy-industrial area, or truck stop.

Figure 46-1C(1), Public-Road Approach Types and Corresponding Design Vehicles, summarizes each type of public-road approach and the corresponding appropriate design vehicles it can accommodate.

### 46-1.05(03) Determining Pavement Section

If, for a public-road approach type A, B, or C, the AADT is 1000 or less, or for a public-road approach type D, the ADTT of FHWA Class 5 trucks is 50 or less, the minimum pavement section shown on the INDOT Standard Drawings should be specified.

If, for a public-road approach type A, B, or C, the AADT is greater than 1000, or for a public-road approach type D, the ADTT of FHWA Class 5 trucks is greater than 50, ESALs must be determined as described in Section 52-8.03(01).

For an HMA approach, the required mix type is determined based on ESALs as shown in Figure 52-9B. The courses and densities should be those identified in the minimum pavement section shown on the INDOT Standard Drawings.

For a PCCP approach, the pavement thickness is determined as described in Section 52-8.03(03).
46-1.06 Intersection Spacing

If creating a new intersection, the designer must ensure that there is sufficient distance between the new and adjacent intersections so that they form distinct intersections. A short distance between intersections should be avoided, if practical, because it tends to impede traffic operations. For example, if two intersections are close together and require signalization, they may need to be considered as one intersection for signalization purposes. To operate safely, each leg of the intersection may require a separate green cycle, thereby greatly reducing the capacity for both intersections. To operate efficiently, signalized intersections should be at least 1300 ft apart. New intersections should preferably be at least 400 ft apart.

A short gap between opposing T intersections should be avoided. A driver tends to encroach into the opposing lanes (corner cutting) to make the turn in one movement.

46-1.07 Design Vehicle

46-1.07(01) Types

The design vehicles used for intersection design are as follows.

1. **P** Passenger car, light panel truck, or pickup truck
2. **SU** Single-unit truck
3. **CITY-BUS** City transit bus
4. **S-BUS-36** Conventional school bus (65 passengers)
5. **A-BUS** Articulated bus
6. **WB-40** Intermediate semitrailer combination
7. **WB-50** Intermediate semitrailer combination
8. **WB-62** Interstate-route semitrailer combination
9. **WB-65** (Indiana Design Vehicle, or IDV) Interstate-route semitrailer combination
10. **WB-100T** Semitrailer combination with three trailers
11. **WB-109D** Turnpike semitrailer combination with two trailers
12. **MH** Recreational vehicle: motor home
13. **P/T** Recreational vehicle: passenger car and camper trailer
14. **P/B** Recreational vehicle: passenger car and boat trailer
15. **MH/B** Recreational vehicle: motor home and boat trailer

See Figure 46-1D, Typical Semitrailer Combination Design Vehicle illustrates a turning path of a semitrailer design vehicle. Section 46-12.0 provides turning templates for the design vehicles which are used by the Department.
**46-1.07(02) Selection**

The selected design vehicle should be based on the largest vehicle that will use the intersection with some frequency. Figure 46-1E, Suggested Design-Vehicle Selection (Intersection), identifies the desirable and minimum design vehicle based on the functional classification of the intersecting highways which the vehicle is turning from and onto.

Some portions of an intersection may be designed with one design vehicle and other portions with another vehicle. For example, it is desirable to design physical characteristics such as curbs or islands for the IDV but to provide painted channelization markings for a passenger car. This will provide a positive indicator for the more-frequent-turning vehicle.

The SU vehicle is the smallest vehicle used in the design of an intersection. This reflects that in a residential area, a delivery truck will be negotiating turns with some frequency. On a facility accommodating regular truck traffic, one of the semitrailer combinations should be used for design. For design purposes, the IDV is permitted to operate on each public highway.

The WB-100T and WB-109D design vehicles are only permitted to operate on the Indiana Toll Road or within 15 mi of its toll gates.

**46-2.0 TURNING RADIUS FOR RIGHT TURN**

The turning-radius treatment for an intersection at-grade influences the operation, safety, and construction costs of the intersection. Turning-radius design may not receive sufficient attention. Therefore, the designer should ensure that the design is compatible with the intersection operation. Section 46-2.01 provides guidance in determining an acceptable turning-radius design. Section 46-2.03 provides the turning-radius design which may be used for preliminary design purposes.

**46-2.01 Design for Pavement Edge or Curb Line**

Once the designer has selected the design vehicle (Section 46-1.07), the proper pavement-edge or curb-line location must be determined, as described below.

**46-2.01(01) Inside Clearance**

The selected design vehicle will make a right turn while maintaining approximately a 2-ft clearance from the pavement edge or curb line and, at a minimum, will not come closer than 1 ft.
46-2.01(02) Encroachment

To determine the acceptable encroachment, the designer should evaluate factors including traffic volume, one-way or two-way operation, urban or rural location, and functional classifications of the intersecting roads or streets. The following will apply.

1. **Urban.** The selected design vehicle should not encroach into the opposing travel lanes. However, this is not always practical or cost effective. Figure 46-2A provides recommended criteria for acceptable encroachment for a right-turning vehicle at an urban intersection. The designer must evaluate these encroachment recommendations against the construction and right-of-way impacts. For example, if the impacts are significant and if through or turning volume is relatively low, the designer may decide to accept an encroachment of the design vehicle which exceeds the criteria in shown Figure 46-2A.

2. **Rural.** The selected design vehicle should not encroach onto the adjacent lane on the road from which the turn is made nor into the opposing lanes of traffic onto the road which the turn is made.

If there are two or more lanes of traffic in the same direction on the road onto which the turn is made, the selected design vehicle can occupy both travel lanes. The turning vehicle should be able to make the turn while remaining entirely in the right through lane.

46-2.01(03) Parking Lane or Shoulder

A parking lane or shoulder will be available on one or both approach legs, and this additional roadway width may be carried through the intersection. This will greatly ease the turning problem for a large vehicle at an intersection with a small curbed radius. Figure 46-2B illustrates the turning paths of design vehicles where the radius is 15 ft or 25 ft and where an 8- to 10-ft parking lane is provided. The presence of a shoulder of 8 to 10 ft width will have the same impact as a parking lane.

The figure also illustrates the necessary distance to restrict parking before the PC (15 ft) and after the PT (20 to 40 ft) on the cross street. The designer will, of course, need to check the proposed design with the applicable turning template and encroachment criteria. The designer should not consider the beneficial effects of a parking lane if the lane will be used for through traffic for part of the day.

Where the turning volume is low, the typical shoulder pavement structure may be used. However, where the turning volume is high or where there is a significant number of turning trucks, a full-depth
shoulder pavement should be constructed. Figure 46-2B also indicates where the parking lane or shoulder should have a full-depth pavement structure. This treatment is critical to avoid pavement deterioration from trucks turning at the intersection.

46-2.01(04) Pedestrians

The greater the turning radius, the farther a pedestrian must walk across the roadway. This is especially important for pedestrians with disabilities or a handicapped individual. Therefore, the designer should consider the number of pedestrians when determining the pavement-edge or curb-line location. This may lead to, for example, the decision to use a simple curve with taper offsets or a turning roadway (see Section 46-3.0) to provide a pedestrian refuge.

46-2.01(05) Type of Turning Design

Once the designer has determined the basic turning parameters (e.g., design vehicle, encroachment, inside clearance), it is necessary to select a type of turning design for the curb return or pavement edge which will be in accordance with the criteria described below and will fit the intersection constraints.

The simple radius is the easiest to design and construct and is used at an urban intersection. However, the simple radius with an entering and exiting taper provides a better fit to the transitional turning path of a vehicle. The simple radius with tapers should be used at a rural intersections, or, desirably, at an urban intersection. A simple radius without tapers may be used for an urban intersection design. Advantages of the simple radius with exiting and entering tapers as compared to the simple radius without tapers are as follows.

1. To accommodate a specific design vehicle, a simple radius with tapers requires less intersection pavement than a simple radius without tapers. Another benefit is the reduced right-of-way impact at the intersection corners. For a large vehicle, a simple radius is often an unreasonable design, unless a channelized island is used and, in effect, a turning roadway is installed.

3. A simple radius without tapers results in a greater distance for a pedestrian to cross than a simple radius with tapers.

4. For an angle of turn greater than 90 deg, a simple radius with tapers is a better design than a simple radius without tapers, primarily because less intersection area is required.
46-2.01(06) Turning Template

To determine the final design, the designer must use a turning template for the selected design vehicle. The template will be applied to the intersection to determine how best to meet the criteria for turning-radius design.

46-2.02 Summary

Figure 46-2C illustrates the factors which should be evaluated in determining the proper design for a right turns at an intersection. In summary, the following procedure applies.

1. Select the design vehicle; see Figure 46-1E.

2. Determine the acceptable inside clearance; see Section 46-2.01(01).

3. Determine the acceptable encroachment; see Section 46-2.01(02).

4. Consider the benefits of a parking lane or shoulder; see Section 46-2.01(03).

5. Consider impacts on pedestrians; see Section 46-2.01(04).

6. Select the type of turning treatment see Section 46-2.01(05) (simple radius or simple radius with entering and exiting tapers).

7. Check the proposed design with the applicable vehicular turning template.

8. Revise the design as necessary to accommodate the right-turning vehicle or determine that it is not practical to meet this design because of adverse impacts.

46-2.03 Turning-Radius Design

Figure 46-2D provides recommended the minimum turning radius for various design vehicles, angles of turn, and acceptable encroachment which may be used in the preliminary design. Figure 46-2E illustrates the assumptions used to develop these figures. As an alternative, the designer may want to consider using the public-road approach details in the INDOT Standard Drawings. For the final design, the designer should check the intersection layout using the procedures described in Section 46-2.01.
46-3.0 TURNING ROADWAY

A turning roadway is a channelized area (separated by an island) at an intersection at-grade which allows for a moderate-speed, free-flowing right turn. An interchange ramps is not considered to be a turning roadway.

46-3.01 Guidelines

1. **Area Classification.** A turning roadway is provided in the areas as follows.
   
   a. **Rural.** At the intersection of two rural arterials, a turning roadway is provided for each right-turn movement. At the intersection of other functionally-classified roadways, the need for a turning roadway will be determined as required.

   b. **Urban.** A turning roadway is provided at the intersection of two urban arterials if they are within the suburban or intermediate sub-classifications. Because a turning roadway requires more right of way than a simple intersection, its use will rarely be practical in a built-up area. At the intersection of other functionally-classified roadways, the need for a turning roadway will be determined as required.

2. **Speed.** A turning roadway is desirable if the turning speed is 10 mph or higher.

3. **Angle of Turn.** A turning roadway should be considered if the angle of turn is greater than 90 deg. An intersection with an angle of turn of less than 90 deg does not lend itself to the use of a turning roadway.

4. **Island Size.** If there is a significant amount of unused pavement, the designer should consider using a turning roadway. The island size should be at least 100 ft$^2$. The minimum island size in a rural area should be at least 75 ft$^2$, or in an urban area should be at least 50 ft$^2$. If the island will provide a refuge area for pedestrians, the minimum island size should be at least 160 ft$^2$.

5. **Island Type.** An island of 75 ft$^2$ or greater should be constructed as raised and corrugated, and delineated with paint or raised pavement markings, or color-contrast pavement. An island of less than 75 ft$^2$ should only be painted.

6. **Traffic Volume.** A turning roadway should be considered if, during the design hour, there are 50 or more right-turning vehicles from a 2-lane facility or 100 or more right-turning vehicles from a 4-lane facility. The design hour is considered to be 20 years in the future.
7. **Level of Service.** Installation of a turning roadway can often improve the level of service through the intersection. At a signalized intersection, a turning roadway may significantly improve the capacity of the intersection by not requiring the right-turning vehicles to obey the signal. Level-of-service criteria are provided in the geometric-design figures in Chapters Fifty-three and Fifty-five.

8. **Crash.** A turning roadway should be considered if there are significant numbers of rear-end type crashes at an intersection. A turning roadway permits a vehicle to make the turning movement at a higher speed and, consequently, should reduce this type of accident.

9. **Pedestrians.** If pedestrian volume is high, a turning roadway provides a refuge area for a pedestrian crossing a wide intersection.

10. **Truck.** A turning roadway should be considered if the selected design vehicle is a semi-trailer combination.

11. **Width.** A turning-roadway width should not be less than 14 ft.

Figure 46-3A illustrates a typical design for a turning roadway. The figure illustrates a turning roadway with a simple-curve radius with entering and exiting tapers. A turning roadway with a simple curve radius without entering and exiting tapers is also acceptable.

### 46-3.02 Design Criteria

#### 46-3.02(01) Design Speed

The design speed on a turning roadway should be within 20 mph of the mainline design speed. However, a turning roadway with a low design speed (e.g., 10 mph) will still provide a significant benefit to the turning vehicle regardless of the speed on the approaching highway. The design speed for a turning roadway will therefore be in the range of 10 to 20 mph.
46-3.02(02) Width

The turning-roadway width is dependent upon the turning radius and design vehicle selected. Figure 46-1E provides the criteria for selection of the appropriate design vehicle. Figure 46-3B provides turning-roadway pavement width for various design vehicles based on one-lane, one-way operation with no provision for passing a stalled vehicle. The pavement width shown in Figure 46-3B provides an extra 6-ft clearance beyond the design vehicle's swept path. This additional width provides extra room for maneuverability and driver variances.

In selecting the turning roadway width, the designer should also consider the possibility that a larger vehicle may also use the turning roadway. The extra 6-ft clearance shown in Figure 46-3B will allow for the accommodation of the occasional larger vehicle, although at a lower speed and with less clearance. For example, a turning-roadway for a WB-50 vehicle with a 10-ft radius will still accommodate an occasional WB-62 vehicle. However, it would not accommodate a WB-65 vehicle. If there are a significant number of larger vehicles using the turning roadway, it should be selected as the design vehicle.

A shoulder is provided on the right side of a right-turning roadway in a rural area. The width of the shoulder should be the same as the preceding mainline shoulder. However, at a restricted intersection, a narrower shoulder or none may be provided. Where a shoulder is provided, a full-depth shoulder pavement should be constructed.

Additional information on turning-roadway width can be found in AASHTO’s A Policy on Geometric Design of Highways and Streets, such as one-lane, one-way operation with provision for passing a stalled vehicle by another of the same type, or two-lane operation).

46-3.02(03) Pavement Thickness

The entire turning-roadway width, including shoulders and curb offsets, should have a uniform pavement thickness. See Chapter Fifty-two for additional information on pavement design.

46-3.02(04) Horizontal Alignment

The horizontal alignment differs from that of the open-roadway condition, which is discussed in Chapter Forty-three. In comparison, a turning-roadway design is less restrictive, which reflects more-restrictive field conditions, and less-demanding driver expectation and driver acceptance of design limitations. The following discusses the assumptions used to design horizontal alignment for a turning roadway.
1. **Curvature Arrangement.** A simple curve with an entrance and exit taper is the typical curvature arrangement.

2. **Superelevation.** A turning roadway is relatively short in length. This greatly increases the difficulty of superelevating the roadway. Therefore, a flexible approach is used for a superelevated turning roadway. Figure 46-3C provides a range of superelevation rates that the designer may select for the appropriate combination of curve radius and design speed. For a turning roadway with a design speed of 10 to 20 mph, the superelevation rate will be 2%, the normal cross slope. The maximum superelevation rate for a turning roadway should not exceed 6%. Selection of the appropriate superelevation rate will be based on field conditions.

3. **Superelevation Transition.** If a turning roadway is superelevated, the transition length should be in accordance with the criteria shown in Chapter Forty-three for the relative longitudinal slope. For an open roadway, the relative slope is measured between the centerline of the roadway and either pavement edge. The relative longitudinal slope is measured between the left edge of the turning roadway and the right pavement edge. For a turning roadway, the axis of rotation is about the left edge of the traveled way.

   Due to the restrictive nature of a turning roadway and its short length, the minimum transition length will be determined as required. The designer should review the field conditions, deceleration and acceleration taper lengths, right-of-way restrictions, and construction costs to produce a practical design for the superelevation-transition length at a turning roadway.

4. **Superelevation Development.** Figure 46-3D illustrates a schematic of superelevation development at a turning roadway. The actual development will depend upon the practical field conditions combined with a reasonable consideration of the theory behind horizontal curvature. The criteria to be considered are as follows.

   a. No change in the normal cross slope is necessary up to Section B-B. Here, the width of the turning roadway is about 2 ft.

   b. The full width of the turning roadway should be attained at Section D-D. The amount of superelevation at D-D will depend upon the practical field conditions.

   c. Beyond Section D-D, the turning-roadway pavement should be rotated as needed to provide the required superelevation for the design speed of the turning roadway.

   d. The minimum superelevation-transition length should meet the criteria set forth in Item 3 above.
e. The superelevation treatment for the exiting portion of the turning roadway should be similar to that described for the entering portion. However, for a merge at a stop-controlled intersection, the superelevation on the turning roadway should match the cross slope on the merging highway or street.

See the associated discussion shown in the AASTHO *Policy on Geometric Design of Highways and Streets* for more information regarding the specific situations as follows:

a. turning roadway leaves a through road that is on tangent;

b. turning roadway and through lanes curve in same direction;

c. turning roadway and through lanes curve in opposite directions; and

d. there is a speed-change lane.

5. **Minimum Radius.** The minimum turning-roadway radius is based on design speed, side-friction factor, and superelevation (see Chapter Forty-three). Figure 46-3E provides the minimum radius for various turning-roadway conditions. As discussed in Item 2, a range of superelevation rates is available.

6. **Cross-Slope Rollover.** Figure 46-3F provides the maximum allowable algebraic difference in the cross slopes between the mainline and turning roadway where these are adjacent to each other. In Figure 46-3D, these criteria apply between Section A-A and Section D-D. This will be a factor only where a superelevated mainline is curving to the left.

7. **Stopping Sight Distance.** The value for stopping sight distance for the open highway condition is applicable to a turning-roadway intersection of the same design speed. The value shown in Figure 42-1A, together with the value for a design speed of 10 mph, are shown in Figure 46-3E(1).

The sight distance should be available at all points along a turning roadway. Where practical, a longer sight distance should be provided. It applies as a control in the design of both horizontal and vertical alignments.

For a design speed lower than 40 mph, a sag vertical curve, as governed by headlight sight distance, theoretically should be longer than a crest vertical curve. Because the design speed of a turning roadway is governed by the horizontal curvature, and the curvature is relatively
sharp, a headlight beam parallel to the longitudinal axis of the vehicle ceases to be a control. Where practical, a longer length for either a crest or sag vertical curve should be used.

The sight-distance control as applied to horizontal alignment has an equal, if not greater effect on design of a turning roadway than vertical control. The sight line along the centerline of the inside lane around the curve, clear of obstruction, should be such that the sight distance measured on an arc along the vehicle path equals or exceeds the stopping sight distance shown in Figure 46-3E(1). A likely obstruction may be a bridge abutment or line of columns, wall, cut sideslope, or a side or corner of a building.

46-3.02(05) Deceleration or Acceleration Lane

A deceleration or acceleration lane is desirable where a turning roadway is used. However, it may not always be practical when considering field conditions, right-of-way restrictions, and construction costs. The following should be considered in determining the need for a deceleration or acceleration lane with a turning roadway.

1. Turning-Roadway Design Speed. The use of a deceleration or acceleration lane should be considered where the turning roadway design speed is more than 20 mph lower than that of the mainline.

2. Mainline Design Speed. A deceleration or acceleration lane should be considered if the mainline design speed is 50 mph or higher.

3. Traffic Volume. An acceleration or deceleration lane should be considered where the following conditions exist.

   a. Two-Lane Facility. An acceleration or deceleration lane should be considered where the mainline AADT is 5000 or more and there are 75 or more turning vehicles during the design peak hour.

   b. Four-Lane Facility. An acceleration or deceleration lane should be considered where the mainline AADT is 10,000 or more and there are 125 or more turning vehicles during the design peak hour.

4. Storage Length. A deceleration lane may be beneficial at a signalized intersection where the through-lane storage may limit access to the turning roadway. The designer should consider a deceleration lane which extends upstream beyond the storage requirements of the intersection to allow access for a right-turning vehicle into the turning roadway (see Figure 46-3G).
5. **Traffic Condition.** An acceleration lane should be provided if the merging-traffic condition is free-flowing. An acceleration lane should not be considered for a yield- or stop-control condition.

The length of a deceleration or acceleration lane is based on the design speed of the turning roadway and the design speed of the mainline. The length should be in accordance with Section 48-4.0 for a ramp at an interchange.

### 46-3.02(06) Pavement Markings

Figure 46-3H illustrates the pavement-marking details for a turning roadway. For additional information on pavement markings, the designer should review Chapter Seventy-six.

### 46-4.0 RIGHT- OR LEFT-TURN LANE

Where the turning maneuver for a left- or right-turning vehicle occurs in a through travel lane, it disrupts the flow of through traffic. To minimize potential conflicts, the use of a turn lane may be warranted to improve the level of service and safety at the intersection.

### 46-4.01 Turn-Lane Warrants

#### 46-4.01(01) Warrants for a Right-Turn Lane

The use of a right-turn lane can significantly improve operations. An exclusive right-turn lane should be considered as follows.

1. at an unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria shown in Figure 46-4A;

2. at an unsignalized intersection on a high-speed 4-lane urban or rural highway which satisfies the criteria shown in Figure 46-4B;

3. at an intersection where a capacity analysis determines that a right-turn lane is necessary to meet the level-of-service criteria;
4. for uniformity of intersection design along the highway if other intersections have right-turn lanes; or

5. at an intersection where the accident experience, existing traffic operations, sight-distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to a right-turning vehicle.

**46-4.01(02) Warrants for a Left-Turn Lane**

The accommodation of left turns is often the critical factor in proper intersection and median-opening design. A left-turn lane can significantly improve both the level of service and intersection safety. An exclusive left-turn lane should be provided as follows:

1. at each intersection on an arterial, where practical;

2. at each intersection on a divided urban or rural highway with a median wide enough to accommodate a left-turn lane, provided that adequate spacing exists between intersections;

3. at a unsignalized intersection on a 2-lane urban or rural highway which satisfies the criteria shown in Figure 46-4C, Volume Guidelines for Left-Turn Lane on a Two-Lane Highway;

4. at an intersection where a capacity analysis determines that a left-turn lane is necessary to meet the level-of-service criteria, including multiple left-turn lanes;

5. at a signalized intersection where the design-hour left-turning volume is 60 veh/h or more for a single turn lane, or where a capacity analysis determines the need for a left-turn lane;

6. for uniformity of intersection design along the highway if other intersections have left-turn lanes in order to satisfy driver expectancy;

7. at an intersection where the accident experience, traffic operations, sight distance restrictions (e.g., intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to left-turning vehicles; or

8. at a median opening where there is a high volume of left turns, or where vehicular speeds are 50 mph or higher.
46-4.02 Design of Left- or Right-Turn Lane

46-4.02(01) Turn-Lane Width

The width of the turn lane should be determined relative to the functional classification, urban or rural location, and project scope of work. Chapters Fifty-three and Fifty-five provide the applicable width for an auxiliary lane. Those chapters provide criteria for the applicable shoulder width adjacent to an auxiliary lane.

46-4.02(02) Turn-Lane Length

The length of a right- or left-turn lane at an intersection should allow both safe vehicular deceleration and storage of turning vehicles outside of the through lanes. However, it is often not practical to provide a turn-lane length which provides for deceleration. Therefore, the full-width length will often only be sufficient for storage.

The length of an auxiliary lane will be determined by some combination of its taper length, \( L_T \), deceleration length, \( L_D \), and storage length, \( L_S \), and by the mainline functional classification. Figure 46-4H, Functional Length of Auxiliary Turn Lane, provides the length considerations for each functional classification. See Figure 46-4I, Typical Auxiliary Lane at an Intersection. The following will apply.

1. **Taper.** For tangent approaches, the Department’s practice is to use a 100-ft straight-line taper at the beginning of a single turn lane, or a 150-ft straight-line taper at the beginning of dual turn lanes for an urban street. On a curvilinear alignment, the entrance taper should be designed with a constant rate of divergence throughout the curve. The entrance taper length should be at least 50 ft.

2. **Deceleration.** For a rural facility, the deceleration distance, \( L_D \), should meet the criteria shown in Figure 46-4J, Deceleration Distance for Turning Lane. The values determined from Figure 46-4J should be adjusted for grades. Figure 46-4J also provides the grade-adjustment factor. This distance is desirable on an urban facility. However, this is not always feasible. Under restricted urban conditions, deceleration may have to be accomplished entirely within the travel lane. For this situation, the length of turn lane will be determined solely on the basis of providing adequate vehicle storage, i.e., \( L_D = 0 \) ft.

3. **Storage Length for Signalized Intersection.** The storage length, \( L_S \), for a turn lane should be sufficient to store the number of vehicles likely to accumulate in a signal cycle during
the design hour. The following should be considered in determining the recommended storage length for a signalized intersection.

a. The storage length should be based on the cycle length and the traffic volumes during the design hour. For a cycle of less than 120 s, the storage length should be based on 2 times the average number of vehicles that would store during the cycle during the design hour. For a cycle of 120 s or longer, the storage length should be based on 1.5 times the average number of vehicles that would store during the cycle during the design hour. Average vehicle length is assumed to be 20 ft. At a minimum, space should be provided for two passenger cars.

b. Figure 46-4K(1), Recommended Storage Length for Signalized Intersection, illustrates an alternative method to determine the recommended storage length for a left-turn lane, or a right-turn lane where a turn on red is prohibited, for a signalized intersection for which the v/c ratio is known. The values obtained from the figure are for a cycle length of 75 s and a v/c ratio of 0.80. For other values, the length obtained in the figure should be multiplied by the appropriate adjustment factor shown in Figure 46-4K, Storage Length Adjustment Factor. The v/c ratio is determined from a capacity analysis as described in the Highway Capacity Manual.

c. Where a turn on red is permitted or where a separate right-turn signal phase is provided, the length of the right-turn lane may be reduced due to less accumulation of turning vehicles.

4. **Storage Length for Unsignalized Intersection.** The storage length should be sufficient to avoid the possibility of a left-turning vehicle stopping in the through lanes and waiting for a gap in the opposing traffic flow. The minimum storage length should have sufficient length to accommodate the expected number of turning vehicles likely to arrive in an average 2-min period within the design hour. At a minimum, space should be provided for two passenger cars. If truck traffic exceeds 10%, space should be provided for at least one passenger car and one truck. See Figure 46-4L, Recommended Storage Length (Ls) for Unsignalized Intersection.

5. **Minimum Turn-Lane Length.** Under restricted conditions, the minimum full-width right- or left-turn lane length, including deceleration and storage, may be 50 ft where there are less than 10% trucks, or 100 ft where there are 10% or more trucks. This is exclusive of the taper. See Item 1 above for minimum taper length.
At a signalized intersection, the right- or left-turn lane length should exceed the storage length of the adjacent through lane. Otherwise, a vehicular queue in the through lane will block entry into the turn lane for turning vehicles.

46-4.02(03) Channelized Left-Turn Lane

If a left-turn lane is required on a 2-lane highway, it should be designed as a channelized left-turn lane as illustrated in Figure 46-4M, Channelized Left-Turn Lane for 2-Lane Highway. As an alternative, based on site conditions and turning volume, a passing blister may be used at a T intersection. See Section 46-4.03.

46-4.02(04) Slotted Left-Turn Lane

On a 4-lane facility with a wide median, slotted left-turn lanes are desirable where the median width is equal to or greater than 24-ft. The advantages are as follows:

1. better visibility of opposing through traffic;
2. decreased possibility of conflict between opposing left-turning vehicles; and
3. more left-turning vehicles are served.

Figure 46-4N illustrates typical parallel and tapered slotted left-turn lanes. The designer should consider the following.

1. **Slot Length.** The slotted section of the turn lane should be at least 50 ft long with a minimum of 100 ft. The slotted section should not include the required deceleration distance for the turn lane.

2. **Nose Width.** The nose of the slotted lane should be a minimum of 4 ft plus shoulder- or curb-offset width (or return taper) from the opposing through lanes. The nose position should be checked for interference with the turning paths from the cross street.

3. **Slot Angle.** The angle of the slot should not diverge more than 10 deg from the through mainline alignment.
4. **Island.** To delineate the slotted portion, the channelized island for the slotted lane should be a raised corrugated island. Raised pavement markers may be used for further delineation.

**46-4.02(05) Turn-Lane Extension**

On a 2-lane highway, it may be desirable to extend the right-turn lane, if provided, beyond the intersection to allow mainline vehicles to bypass left-turning vehicles on the right. See Section 46-4.03 for passing-blister design at a three-legged intersection. Upon determining the need for a turn lane extension, the designer should consider the following.

1. **Traffic Volume.** A turn-lane extension may be provided at the intersection of public road or street with a 2-lane State highway with a design-year AADT of 5000 or greater. For a 2-lane State highway with a design-year AADT of less than 5000, a turn-lane extension should be used only if one or more of the following occurs.

   a. There is an existing turn lane extension.

   b. There are 20 or more left-turning vehicles during the design hour.

   c. Accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles.

   d. Shoulders indicate heavy use (e.g., dropped shoulders, severe pavement distress).

2. **Scope of Work.** A turn-lane extension should only be used in conjunction with a 3R or partial 3R project. For a new or reconstruction project, a channelized left-turn lane should be provided; see Figure 46-4M.

3. **Right-Turn-Lane Warrant.** A turn-lane extension may be appropriate at a four-legged intersection if a right-turn lane is not warranted.

4. **Design.** If designing a turn-lane extension, the designer should consider the following.

   a. Geometrics. The beginning of the turn lane should be designed as a right-turn lane including width and length (taper, deceleration, and storage). The extension beyond the intersection should be designed as a 300-ft tapered acceleration lane; see Section 46-6.0. Under restricted conditions, the turn-lane extension length may be shortened to meet field conditions, but not to less than 200 ft.
2. Pavement. The turn lane and turn-lane extension should have the same color and pavement texture as the through lanes. The shoulder adjacent to the turn-lane extension should be of contrasting color and texture. The turn-lane extension pavement should be a full-depth shoulder with an additional 200 ft of full-depth shoulder provided after the exiting taper.

c. Sight Distance. Decision sight distance should be provided on the mainline to the intersection to allow a mainline driver enough time to consider whether to pass the left-turning vehicle or come to a stop. Sufficient sight distance should be available so that a side-street driver will not encroach into the auxiliary lane. A stop line should be provided to delineate the proper location for stopping.

d. Concerns. Consideration should be given at an offset intersection to ensure that the turn-lane extension will not lead to operational problems. Distractions such as location of a drive, commercial background lighting, or highway lighting should also be considered in the design.

5. Channelized Left-turn Lane. The decision on whether to use either a channelized left-turn lane or a turn-lane extension should be based on accident history, right-of-way availability, through- and turning-traffic volumes, design speed, and available sight distance. A channelized left-turn lane should be provided if the left-turning volume is high enough that a left-turn lane is warranted as discussed in Section 46-4.01.

46-4.03 Passing Blister

At a three-legged intersection, it may be desirable to provide a passing blister to relieve congestion due to left-turning vehicles. The designer should review the following when determining the need for a passing blister.

1. Traffic Volume. A passing blister may be provided at the intersection of a public road or street with a 2-lane State highway with a design-year AADT of 5000 or greater. For a 2-lane State highway with a design-year AADT of less than 5000, a passing blister should be used only if one or more of the following occurs.

   a. There is an existing passing blister.

   b. There are 20 or more left-turning vehicles during the design hour.
c. Accident reports or site evidence, such as skid marks in the through lane displaying emergency braking, indicate potential problems with left-turning vehicles.

d. The shoulder indicates heavy use (e.g., dropped shoulder, severe pavement distress).

2. **Design.** Figure 46-4 O illustrates and provides the design criteria for a passing blister. An alternative design should be considered if successive passing blisters overlap each other or are within close proximity to each other.

3. **Channelized Left-turn Lane.** The decision on whether to use either a channelized left-turn lane or a passing blister should be based on accident history, right-of-way availability, through- and turning-traffic volumes, design speed, and available sight distance. A channelized left-turn lane should be provided if the left-turning volume is high enough that a left-turn lane is warranted as discussed in Section 46-4.01.

### 46-4.04 Multiple Turn Lane

#### 46-4.04(01) Warrants

Multiple right- or left-turn lanes should be considered as follows:

1. there is insufficient space to provide the necessary length of a single turn lane because of restrictive site conditions (e.g., closely spaced intersections); or

2. based on a capacity analysis, the necessary time for a protected left-turn phase for a single lane becomes unattainable to meet the level-of-service criteria (average delay per vehicle).

Two right-turn lanes do not function as well as two left-turn lanes because of the more restrictive turning movement for a two-abreast right turn. If practical, the designer should find an alternative means to accommodate the high number of right-turning vehicles. For example, a turning roadway may be more efficient.

At an intersection with a very high volume of turning vehicles, two right-turn lanes and three left-turn lanes may be considered. However, multiple turn lanes may cause problems with right of way, lane alignment, crossing pedestrians, and lane confusion for approaching drivers. Therefore, if practical, the designer should consider an alternative design, such as indirect left turns or an interchange.
46-4.04(02) Design

For multiple turn lanes to function properly, design elements must be evaluated. Figure 46-4P illustrates both multiple right- and left-turn lanes. The designer should consider the following.

1. **Throat Width.** Because of the off-tracking characteristics of a turning vehicle, the normal width of two turning lanes may be inadequate to properly receive two vehicles turning abreast. Therefore, the receiving throat width may need to be adjusted. The throat width should be determined from the application of the turning template for the design vehicle (see Item 4 below).

2. **Pavement Markings.** As illustrated in Figure 46-4P, pavement markings can effectively guide two lines of vehicles turning abreast. The Office of Traffic Engineering will determine the selection and placement of pavement markings.

3. **Opposing Left-Turning Traffic.** If simultaneous, opposing multiple left turns will be allowed, the designer should ensure that there is sufficient space for all turning movements. This separation should be 30 ft (See Figure 46-4P). Two left-turn lanes with their two-abreast vehicles can cause problems. Two turning lanes should only be used with signalization providing a separate turning phase.

4. **Turning Template.** The intersection-design elements for multiple turn lanes must be checked by using the applicable turning template. The designer should assume that the selected design vehicle will turn from the outside lane of the multiple turn lanes. The inside vehicle should be a single-unit truck but, as a minimum, the other vehicle can be a passenger vehicle turning side by side with the selected design vehicle.

46-5.0 TWO-WAY LEFT-TURN LANE (TWLTL)

A two-way left-turn lane (TWLTL) is a cost-effective method to accommodate a continuous left-turn demand and to reduce delay and accidents. This type of lane will often improve operations on a roadway which was originally intended to serve the through movement but now must accommodate the demand for accessibility created by changes in adjacent land use.

46-5.01 Guidelines

The following provides guidelines for where a TWLTL should be considered.
1. **General.** The physical conditions under which a TWLTL should be considered include the following:

   a. an area with at least 50 drives per mile, total for both sides;

   b. an area of high-density commercial development; or

   c. an area with substantial mid-block left turns.

   The applicability of the TWLTL is a function of the traffic conditions resulting from the adjacent land use. The designer should evaluate the area to determine the relative attractiveness of a TWLTL as compared to an alternative access technique. For example, a TWLTL may perpetuate more strip development. If this is not desirable, a raised median may be more appropriate.

2. **Functional Classification.** An undivided 2-lane or 4-lane urban or suburban arterial is the most common candidate for the implementation of a TWLTL. This is commonly referred to as a 3-lane or 5-lane facility, respectively. The use of a TWLTL on a 6-lane arterial (i.e., a 7-lane facility) is not appropriate. See Section 46-5.03.

3. **Traffic Volume.** Traffic volume is a significant factor in the consideration of a TWLTL. The design year which should be used to determine the traffic volume is 20 years. The following should be considered.

   a. On an existing 2-lane roadway, a TWLTL will be advantageous for AADT between 5,000 and 12,500.

   b. On an existing 4-lane highway, a TWLTL will be advantageous for AADT between 10,000 and 25,000.

   c. For an AADT greater than 25,000, a raised median may be more appropriate. For a 6-lane highway, a raised median is recommended.

   d. Pedestrian-crossing volume is also a consideration because of the large paved area which must be traversed where a TWLTL is present (i.e., no pedestrian refuge exists).
4. **Speed.** The design speed is a major factor in TWLTL application. A design speed of 25 to 50 mph will properly accommodate a TWLTL. For a posted speed limit higher than 50 mph, its use should be considered only as required.

5. **Accident History.** On a high-volume urban or suburban arterial, traffic conflicts often result because of a significant number of mid-block left turns combine with significant opposing traffic volume. This may lead to a disproportionate number of mid-block, rear-end, or sideswipe accidents. A TWLTL is likely to reduce these types of accidents. The designer should review and evaluate the available accident data to determine if unusually high numbers of these accidents are occurring.

46-5.02 **Design Criteria**

46-5.02(01) **Lane Width**

Recommended lane width is shown in Chapter Fifty-three or Fifty-five. An existing highway that warrants the installation of a TWLTL is often located in an area of restricted right of way, so conversion of the existing cross section may be difficult. To obtain the TWLTL width, the designer may have to consider the following:

1. removing an existing raised median;

2. reducing the width of existing through lanes;

3. reducing the number of existing through lanes;

4. eliminating an existing parking lane;

5. eliminating or reducing the width of an existing shoulder; or

6. acquiring additional right of way to expand the pavement width by the amount needed for the TWLTL.

Item 1 or 6 listed above would be the most advantageous alternative. If this is not practical, the designer will have to evaluate the trade-offs between the benefits of the TWLTL and the negative impacts of eliminating or reducing the width of one or more existing cross-section elements. This may involve a capacity analysis or an in-depth evaluation of the existing accident history.
46-5.02(02) Intersection Treatment

A TWLTL must either be terminated in advance of an intersection to allow the development of an exclusive left-turn lane or be extended up to the intersection. Where the TWLTL is extended up to the intersection, the pavement marking will switch from two opposing left-turn arrows to one left-turn arrow only. In determining the intersection treatment, the following should be considered.

1. **Signalization.** The TWLTL should be terminated, as this type of intersection will warrant an exclusive left-turn lane. At an unsignalized intersection, the TWLTL may be extended through the intersection because an exclusive left-turn lane is usually not required.

2. **Turning Volume.** The left-turn demand into the intersecting road is a factor in determining the proper intersection treatment. If the minimum storage length will govern (Section 46-4.02), it will probably be preferable to extend the TWLTL up to the intersection (i.e., provide no exclusive left-turn lane).

3. **Minimum Length of TWLTL.** The TWLTL should have sufficient length to operate properly. The type of intersection treatment will determine the length of the TWLTL. The appropriate minimum length is influenced by the operating speed on the highway. The following guidance may be used.
   
a. On a facility where $V \leq 30$ mph, the minimum uninterrupted length of a TWLTL should be 500 to 1000 ft.
   
b. On a facility where $V > 30$ mph, the minimum uninterrupted length of a TWLTL should be at least 1000 ft.

The final decision on the length of the TWLTL will be based on site conditions.

4. **Operational or Safety Factors.** Extending the TWLTL up to an intersection could result in operational or safety problems. A driver may, for example, pass through the intersection in the TWLTL and turn left just beyond the intersection into a drive which is within 30 ft of the intersection. If operational or safety problems are known or anticipated at an intersection, it may be preferable to remove the TWLTL prior to the intersection (i.e., provide an exclusive left-turn lane).

5. **Pavement Markings.** Figure 46-5A illustrates the typical pavement markings for a TWLTL at an unsignalized or signalized intersection. Chapter Seventy-six provides additional information for marking a TWLTL.
46-5.03 6-Lane Section

For AADT greater than 25,000, the designer should use a 6-lane section with a raised median. The decision on whether to provide a TWLTL instead of a raised median will be determined as required for the project. The following lists the factors the designer should consider in making this determination.

1. A TWLTL tends to be safer under the existing or proposed conditions as follows:
   a. at least 70 drives per mile;
   b. fewer than 2 signalized intersections per mile; and
   c. a maximum of 4 to 6 approaches per mile, depending on the number of signals per mile.

2. There may be insufficient gaps available in the oncoming traffic to allow a vehicle to make a left turn from the TWLTL in an acceptable period of time.

3. A left-turning vehicle from a roadside drive may try to use the TWLTL as an acceleration lane or as a waiting area before merging in with the mainline traffic.

4. There may be significant delays at a signal to handle crossing pedestrians with a TWLTL. A raised median may be able to provide a refuge area for crossing pedestrians.

5. At a signalized intersection requiring two turn lanes, it will be more difficult to develop the additional lane with a TWLTL. There may be more lane confusion at the intersection for the approaching driver.

6. A raised median forces a driver to make all left turns at intersections, which may overload the capacity of the intersection and increase driver travel time.

7. With a raised median, the left-turn movements are concentrated at the intersections, thereby reducing the conflict area of the overall facility.

8. A raised median may discourage patrons from using facilities on the other side of the road (e.g., gas station, convenience store, restaurant).
9. A raised median discourages new strip development, whereas a TWLTL may encourage such development.

46-6.0 INTERSECTION ACCELERATION LANE

It may be necessary to provide an acceleration lane for turning vehicles at an intersection to allow these vehicles to accelerate before merging with the through traffic.

46-6.01 Acceleration Lane for Right-Turning Vehicle

An acceleration lane should be considered as follows:

1. where the intersection is near or at capacity (LOS of E) in the design year;

2. where a turning roadway is used (see Section 46-3.0);

3. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility;

4. where there is a significant history of rear-end or sideswipe accidents;

5. where there is inadequate intersection sight distance available; or

6. where there is a high volume of trucks turning onto the mainline.
46-6.02 Acceleration Lane in Median

An acceleration lane should be considered for left-turning vehicles as follows:

1. where the turning traffic at an unsignalized intersection must merge with a high-speed, high-volume facility. The acceleration lane may reduce the need for a signalized intersection;

2. where there is a significant history of rear-end or sideswipe accidents;

3. where there is inadequate intersection sight distance available; or

4. where there is a high volume of trucks turning onto the mainline.

46-6.03 Design Criteria

1. Types. An acceleration lane is of the parallel design. Chapter Forty-eight provides additional information.

2. Length. A right-turn or median acceleration lane should be in accordance with Chapter Forty-eight. The controlling curve at an intersection is the design speed of the turning roadway or the speed at which a vehicle can make the right or left turn, usually less than 10 mph. For a 2-lane mainline, the truck acceleration length should be considered where there are 20 to 50 or more turning trucks per day. For a 4-lane facility, it should be considered where there are 75 to 100 or more turning trucks per day.

3. Taper. A 300-ft taper should be used at the end of a parallel acceleration lane.

46-7.0 ADDITIONAL THROUGH LANE AT INTERSECTION

To meet the level-of-service criteria for the design year, it may be necessary to add at least one through lane to an intersection approach. However, an additional lane should be extended beyond the intersection to fully realize the capacity benefits. Figure 46-7A provides criteria for determining how far such a lane should be extended beyond the intersection.

The minimum full-width length of the through lane extension, $D_E$, is that distance needed for the stopped vehicle to accelerate to 5 mph below the average running speed of the highway. The full-
width length of the through-lane extension will be the stored-vehicle length which will cross the intersection during a green cycle.

The length shown in Figure 46-7A may or may not be sufficient for the vehicle to merge into the primary through lane. Therefore, Figure 46-7A should only be used for preliminary design purposes. The final design will be based on site conditions and traffic volume.

The taper rate at the end of the additional through lane will be based on the criteria provided in Figure 46-7A. If curbing is used within the taper area, the curbing should be painted to provide better delineation of the taper.

46-8.0 MEDIAN OPENING

46-8.01 Non-Freeway

46-8.01(01) Warrants

A median opening should be provided on a divided non-freeway at each intersection with a public road or major traffic generator (e.g., shopping center). However, this may result in close intersection spacing which impairs the operation of the facility. The following minimum spacings should be evaluated when determining the warrant for a median opening.

1. Rural Intersection. An opening is provided at each public-road intersection. An opening may not be provided at a minor public road.

2. Urban Intersection. A median opening is provided at each intersection. However, to improve capacity and traffic efficiency, the designer may elect not to provide an opening for a traffic generator if there are other points of access within a reasonable distance of the generator.

The minimum spacing between median openings should be 400 ft. The desirable spacing should be 800 ft. See Figure 46-8A. This allows for the development of a future exclusive left-turn lane with the proper taper, deceleration, and storage length.

If determining the median-opening location, the designer also needs to consider the available sight distance (see Section 46-10). An opening with restricted sight distance may require additional design considerations (e.g., traffic signal, closing the opening).
46-8.01(02) Design

Figure 46-8B illustrates the turning path for a semi-trailer design vehicle and other design criteria at a median opening. The following will apply.

1. **Design Vehicle.** The path of each design vehicle making a minimum left turn at 10 to 20 mph should be considered. Where the volume and types of vehicles making the left-turn movement require a speed higher than the minimum, the radius of turn should correspond to the speed deemed appropriate. However, the minimum turning path at low speed is required for minimum design, and for testing a layout developed for one design vehicle when used by an occasional larger vehicle.

2. **Radius.** The following control radius should be used for the design of a median opening.
   a. 40 ft. Accommodates a P vehicle and an occasional SU vehicle sometimes swinging wide.
   b. 50 ft. Accommodates an SU vehicle and an occasional WB-40 vehicle sometimes swinging wide.
   c. 75 ft. Accommodates a WB-40 or WB-50 vehicle with only minor swinging at the end of the turn.

3. **Encroachment.** The design should be to allow the design vehicle to make a left turn and remain entirely within the inside travel lane of the divided facility (i.e., there will be no encroachment into the through lane adjacent to the inside travel lane). It will be acceptable for the design vehicle to occupy both travel lanes in its turn.

4. **Width.** The median width will be determined by the design of the major highway and the available right of way. The designer should consider the following:
   a. If practical, the median width at the intersection should be wide enough to fully protect a stopped passenger car within the median.
   b. A slotted left-turn lane may be used with a median width greater than 24 ft; see Section 46-4.02(04).
   c. The median width should be wide enough to accommodate a left-turn bay and, where necessary, a two-left-turn-lanes bay.
5. **Median-Nose Design.** The median nose will have either a semicircular end or a bullet-nose end. The median-nose design selection is dependent upon the median-opening function and the median width. The INDOT *Standard Drawings* illustrate the layout for a median nose.

6. **Length of Opening.** The length of a median opening should properly accommodate the turning path of the design vehicle. The minimum median-opening length is 40 ft. It should be as great as the width of crossroad’s traveled way plus its shoulders. If the crossroad is a divided highway, the length of opening should be at least equal to the width of the crossroad’s traveled way plus that of the median. However, each median opening should be evaluated individually to determine the proper length of opening. Figures 46-8C, 46-8D, and 46-8E illustrate median-opening criteria for various design vehicles. The designer should consider the following in the evaluation.

   a. Turning Template. The designer should check the proposed design with the turning template for the design vehicle most likely to use the intersection. Consideration should be given to the frequency of the turn and to the encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

   b. Nose Offset. At a 4-leg intersection, traffic passing through the median opening (going straight) will pass the median nose. To provide a sense of comfort for such a driver, the offset between the nose and the through travel lane (extended) should be at least 2 ft.

   c. Lane Alignment. The designer should ensure that lanes line up properly for crossing traffic.

   d. Location of Crosswalk. A pedestrian crosswalk will intersect the median nose to provide some refuge for pedestrians. Therefore, the median-opening design should be coordinated with the location of the crosswalk.

   A median opening longer than 80 ft should be avoided, regardless of skew.

7. **U-turn.** A median opening is sometimes used only to accommodate a U-turn on a divided non-freeway. Where needed, the spacing should be 1300 to 2500 ft. The design for a U-turn maneuver on an arterial should permit the design vehicle to turn from an auxiliary left-turn lane in the median into the lane next to the outside shoulder or outside curb and gutter on the roadway of the opposing traffic lanes. The INDOT *Standard Drawings* provide additional information on the Department’s U-turn median-opening.
8. **Pavement.** The median-opening pavement will be the same material type and design strength as the adjacent mainline. Chapter Fifty-two provides additional information on pavement design.

9. **Drainage.** The designer should ensure that drainage from the mainline is not allowed to flow or pond within the median opening. See Part IV for INDOT roadway drainage criteria.

10. **Skew.** A control radius for the design vehicle as the basis for the minimum design of a median opening results in a length of opening that increases with the skew angle of the intersection. The skew introduces other variations in the shape of the nose. At a skewed crossing, such control radius should be used in the acute angle used to locate the beginning of the nose on the median edge.

    A channelization lane, left-turn lane, or adjustment to reduce the crossroad skew may be required to limit the opening to the maximum length shown in Item 6 above. An asymmetrical bullet nose is preferable.

**46-8.02 Median Opening on Freeway**

On a fully-access-controlled freeway, median crossing is denied to the public. However, an occasional median opening or emergency crossover is required to accommodate a maintenance or emergency vehicle. Section 54-6.0 provides the Department’s criteria for a median opening on a fully-access-controlled facility.

**46-9.0 CHANNELIZING ISLAND**

The treatments described in this Chapter may require a channelizing island within the intersection area (e.g., turning roadway). The design of an island should consider site-specific functions, including definition of vehicular path, separation of traffic movements, prohibition of movements, protection of pedestrians, and placement of traffic control devices.

**46-9.01 Types of Islands**

Islands are grouped into the following functional classes. Most islands serve at least two of these functions.
1. **Directional Island.** A directional island (e.g., for turning roadways) controls and directs traffic movements and guides the driver into the proper channel.

2. **Divisional Island.** A divisional island separates opposing traffic flows, alerts the driver to the crossroad ahead, and regulates traffic through the intersection. Such an island is introduced at an intersection on an undivided highway, and is particularly advantageous in controlling left turns at a skewed intersection.

3. **Refuge Island.** A refuge island at or near a crosswalk aids or protects a pedestrian crossing a wide roadway. Such an island may be required for a pedestrian where complex signal phasing is used.

4. **Protection Island.** A protection island is used for the protection of a traffic control device.

### 46-9.02 Selection of Island Type

A channelizing island may be a combination of flush or raised; concrete, asphalt, or earth; or triangular or elongated. Selection of an appropriate type of island should be based on traffic characteristics, cost considerations, and maintenance needs. The following offers guidance as to where a flush or raised corrugated island is appropriate.

A flush island is appropriate as follows:

1. on a high-speed rural highway to delineate separate turning lanes;
2. in a constrained location where vehicular path definition is desired, but space for a larger, raised island is not available;
3. to separate opposing traffic streams on a low-speed street; or
4. for temporary channelization either during construction or to test traffic operations prior to installation of a raised island.

A raised corrugated island is appropriate as follows:

1. where a primary function of the island is to provide a pedestrian refuge;
2. where a primary or secondary island function is the location of a traffic signal, sign, or other fixed object;
3. where the island is intended to prohibit or prevent a traffic movement;

4. on a low- to moderate-speed highway where the primary function is to separate high volumes of opposing traffic movements; or

5. at a location requiring positive delineation of vehicular path, such as at a major-route turn or an intersection with unusual geometry.

A channelizing island with curbs should not be used.

### 46-9.03 Minimum Size

An island should be large enough to command the driver’s attention. Island shapes and sizes vary from one intersection to another. For a triangular island, the minimum size is 50 ft² (urban) or 75 ft² (rural). Its area should be at least 100 ft². An island used for pedestrian refuge should be at least 150 ft². An elongated island should not be less than 4 ft wide, and should be 20 to 26 ft long. Where space is limited, an elongated island may be reduced to a minimum width of 2 ft. A curbed divisional island introduced at an isolated intersection on a high-speed highway should be at least 100 ft in length.

### 46-9.04 Delineation

Delineation of a small island is effected primarily by a curb. A large curbed island may be sufficiently delineated by color and texture contrast of vegetative cover, mounded earth, shrubs, reflector posts, or a combination of these. In a rural area, an island curb should be sloping. A vertical or sloping curb may be appropriate in an urban area, depending on the conditions.

A channelizing island should be delineated based on its size, location, and function. An island with raised corrugations presents the most positive means of delineation and may be used with any design speed. For the layout of a raised corrugated island, see the *INDOT Standard Drawings*. For a flush island, it may be appropriate to complement the pavement markings with raised reflectors.

Raised pavement markings, raised reflectors, roughened pavement, or paint striping is used in advance of and around the island to warn the driver. These traffic control devices are important at the approach to a divisional curbed island for the direction of approaching traffic. Figures 46-9A and 46-9B illustrate the pavement markings used with a channelizing island. Section 502-2.08 provides additional information for pavement markings around an island.
46-9.05 Island Offset to Through Lanes

In an urban area on an approach roadway without shoulders, the raised corrugated island should be offset 2 ft from the travel lane. Where shoulders are present, the raised corrugated island should be offset a distance equal to the shoulder width. In a rural area or where a separate turning lane is used, the island should be offset from the turning lane by 2 ft (see Figure 46-9A). If there is no turning lane, the island should be offset a distance equal to the shoulder width. If a corner island is preceded by a right-turn deceleration lane, the shoulder offset should be at least 8 ft.

The designer should also ensure that the island will not interfere with the turning movement of a truck turning from the opposite side on a 4-legged intersection. If there is a conflict, the island should be set back farther or made flush.

46-9.06 Typical Channelizing Intersection

Figure 46-9C illustrates an example of an island treatment at an intersection. Each channelizing intersection must be studied individually considering turning volumes, traffic lane configurations, potential conflicts, and practical signing arrangements.

46-10.0 INTERSECTION SIGHT DISTANCE (ISD)

For an at-grade intersection to operate properly, adequate sight distance should be available. The designer should provide sufficient sight distance for a driver to perceive potential conflicts and to perform the actions needed to negotiate the intersection safely.

The additional costs and impacts of removing sight obstructions are often justified. If it is impractical to remove an obstruction blocking the sight distance, the designer should consider providing traffic-control devices or applications (e.g., warning signs, traffic signals, or turn lanes) which may not otherwise be warranted.

The height of eye for a passenger car driver should be taken as 3.5 ft. The height of eye for a single-unit or combination-truck driver should be taken as 7.6 ft. Its height of object should be taken as 3.5 ft.

The sight line is shown on the plans in the plan and profile views. The proposed profile grade line along the centerline is also shown, however, this is meaningless for intersection sight distance analysis. The proposed ground line under the sight line is the relevant line.
** PRACTICE POINTER **

Intersection sight distance should be analyzed for each local service road or frontage road in the same manner as a public road.

46-10.01 No Traffic Control

An intersection between a low-volume and a low-speed road or street should be either yield-controlled or stop-controlled. However, for a local-road or -street intersection with no traffic control, sufficient corner sight distance should be available to allow an approaching vehicle to see a potentially conflicting vehicle in sufficient time to stop before reaching the intersection. Figure 46-10A provides the ISD criteria for an intersection with no traffic control and approach grades between −3% and +3%. For approach grades greater than 3%, multiply the sight distance value in Figure 46-10A by the appropriate adjustment factor from Figure 46-10B. These Figures are not applicable to State highways.

If the appropriate sight distance cannot be provided, consideration should be given to installing a “Stop” sign on one or more approaches.

46-10.02 Yield Control

46-10.02(01) Intersection With Yield Control on the Minor Road

A driver approaching a Yield sign is permitted to enter or cross the major road without stopping, if there is no potentially conflicting vehicle on the major road. The sight distance needed by a driver on a yield-controlled approach exceeds that for a stop-controlled approach.

A yield-controlled approach needs greater sight distance than a stop-controlled approach, especially at a four-leg yield-controlled intersection where the sight distance needs of the crossing maneuver should be considered. If sight distance sufficient for yield control is not available, use of a “Stop” sign instead of a “Yield” sign should be considered. At a location where the recommended sight distance cannot be provided, consideration should be given to installing other traffic control devices at the intersection on the major road to reduce the speeds of approaching vehicles.
46-10.02(02)  Left- or Right-Turn Maneuver

The length of the leg of the approach sight triangle along the minor road to accommodate a left or right turn without stopping should be 80 ft. This distance is based on the assumption that a driver making a left or right turn without stopping will slow to a turning speed of 10 mph.

The leg of the approach sight triangle along the major road is similar to the major-road leg of the departure sight triangle for a stop-controlled intersection. However, the time gap for a left turn, as shown in Section 46-10.03 should be increased by 0.5 s to the value shown in Figure 46-10C. The appropriate length of the sight triangle leg is shown in Figure 46-10D for a passenger car. The minor-road vehicle needs 3.5 s to travel from the decision point to the intersection. This represents additional travel time that is needed at a yield-controlled intersection, but is not needed at a stop-controlled intersection. However, the acceleration time after entering the major road is 3.0 s less for a yield condition than for a stop condition because the turning vehicle accelerates from 10 mph rather than from a stop condition. The net 0.5-s increase in travel time for a vehicle turning from a yield-controlled approach is the difference between the 3.5-s increase in travel time and the 3.0-s reduction in travel time.

A departure sight triangle like that provided for a stop-controlled approach should also be provided for a yield-controlled approach to accommodate a minor-road vehicle that stops at the “Yield” sign to avoid a conflict with a major-road vehicle. However, because the approach sight triangle for a turning maneuver at a yield-controlled approach is larger than the departure sight triangle used at a stop-controlled intersection, no specific check of departure sight triangle at a yield-controlled intersection should be needed.

A yield-controlled approach needs greater sight distance than a stop-controlled approach, especially at a four-leg yield-controlled intersection where the sight distance needs of the crossing maneuver should be considered. If sight distance sufficient for yield control is not available, use of a “Stop” sign instead of a “Yield” sign should be considered. At a location where the recommended sight distance cannot be provided, consideration should be given to installing other traffic control devices at the intersection on the major road to reduce the speeds of approaching vehicles.

46-10.02(03)  Turning Roadway

Yield control may also exist, for example, at a freeway ramp terminal where the ramp traffic is provided a free-flowing right turn onto the minor road. The assumptions as discussed in Section 46-
are also applicable to turning-roadway yield conditions, except the eye location of the entering vehicle is on the turning roadway itself (see Figure 46-10E).

If insufficient intersection sight distance is available for the operational characteristics of yield control, it may be appropriate to convert the intersection to stop control.

46-10.03 Stop Control

Where traffic on the minor road of an intersection is controlled by a “Stop” sign, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure. The location of the eye should be as described in Section 46-10.03(01).

46-10.03(01) Departure Sight Triangle and Time Gap

The departure sight triangle for an intersection with stop control on the minor road must consider the situations as follows:

1. left turn from the minor road;
2. right turn from the minor road; and
3. crossing the major road from minor-road approach.

A departure sight triangle for traffic approaching from either the right or left, like that shown in Figure 46-10F, Departure Sight Triangles, should be provided for a left turn from the minor road onto the major road for each stop-controlled approach.

Field observations of the gaps in major-road traffic actually accepted by drivers turning onto the major road have shown that the values shown in Figure 46-10G, Intersection Sight Distance for Stop-Controlled Intersection, provide sufficient time for the minor-road vehicle to accelerate from a stop and complete a left turn without unduly interfering with major-road traffic operations.

The intersection sight distance in both directions should be equal to the distance traveled at the design speed of the major road during a period of time equal to the time gap. At a minimum, ISD should be checked for both a passenger car and a single unit truck turning from the minor-road approach. Where a substantial volume of heavy vehicles enter the major road, the use of combination trucks should be considered.
No adjustment is needed for the major-road grade. However, if the minor-road design vehicle is a truck and the intersection is located near a sag vertical curve with a grade over 3%, an adjustment of the intersection sight distance should be considered.

Figure 46-10G provides the criteria for intersection sight distance in both directions for a vehicle turning left.

Intersection sight distance for a left turn at a divided-highway intersection should consider multiple design vehicles and median width. If the design vehicle used to determine sight distance for a divided-highway intersection is larger than a passenger car, sight distance for a left turn will need to be checked for that selected design vehicle and for smaller design vehicles as well. If the divided-highway median is wide enough to store the design vehicle with a clearance to the through lanes of 3 ft at both ends of the vehicle, no separate analysis for the departure sight triangle for a left turn is needed on the minor-road approach for the near roadway to the left.

If the design vehicle can be stored in the median with adequate clearance to the through lanes, a departure sight triangle to the right for a left turn should be provided for that design vehicle turning left from the median roadway. Where the median is not wide enough to store the design vehicle, a departure sight triangle should be provided for that design vehicle to turn left from the minor-road approach. The median width should be considered in determining the number of lanes to be crossed. The median width should be converted to an equivalent number of lanes.

The sight triangle for a left or right turn onto the major road will also provide more than adequate sight distance for a minor-road vehicle to cross the major road. However, the intersection sight distance for a crossing maneuver must be checked for the situations as follows:

1. where left or right turns are not permitted from a particular approach and the crossing maneuver is the only legal maneuver;

2. where the crossing vehicle would cross the equivalent width of more than 6 lanes; or

3. where a substantial volume of heavy vehicles cross the highway, and steep grades that might slow such vehicles while their back portions are still in the intersection are present on the departure roadway on the far side of the intersection.

The time gap shown in Figure 46-10H(1), Time Gap for Crossing Maneuver, may be used for the crossing-maneuver check.
Figure 46-10H, Intersection Sight Distance for Passenger Car to Turn Right provides the intersection sight distance for a passenger car making a right turn from a stop or a crossing maneuver.

At a divided-highway intersection, depending on the median width and the length of the design vehicle, intersection sight distance may need to be considered for crossing both roadways of a divided highway or for crossing the near lanes only and stopping in the median before proceeding.

The ISD value will establish one leg of the sight triangle which needs to be visible to the entering vehicle. The leg on the stop-controlled road or street will be determined by the assumed location of the eye. This is established as 18 ft behind the edge of the travel lane for a new or reconstruction project, or 14.5 ft for a 3R project (see Figure 46-10F, Departure Sight Triangle).

46-10.03(02) Measures to Improve Intersection Sight Distance

The available ISD should be checked using the parameters described above. If the line of sight falls above a bridge railing and guardrail and the ISD value from Figure 46-10G is provided, no further investigation is needed. If the line of sight is restricted by the bridge railing, guardrail, or other obstruction, or the horizontal and vertical alignment of the major road and the ISD value is not available, one or more of the modifications, or a combination of them, should be evaluated to achieve the intersection sight distance as follows:

1. relocate the minor road or drive farther from the end of the bridge;
2. widen the structure on the side where the railing is restricting the line of sight;
3. flare the approach guardrail;
4. revise the grades on the major road or the minor road or drive;
5. remove the obstruction that is restricting sight distance;
6. close the minor road or drive;
7. make the minor road or drive one-way away from the major road; or
8. review other measures that may be practical at a particular location.

If intersection sight distance along the major road cannot be achieved, consideration should be given to installing advance intersection signing with advisory speed plates.
46-10.04 Left Turn From the Major Road

Each location along the major road from which a vehicle is permitted to turn left across opposing traffic, including an intersection or drive, should have sufficient sight distance to accommodate the left-turn maneuver. A left-turning driver needs sufficient sight distance to decide when it is safe to turn left across the lanes used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance. The sight distance along the major road to accommodate a left turn is the distance traversed at the design speed of the major road in the travel time for the design vehicle shown in Figure 46-10I.

The figure also includes appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle. The unadjusted time gap shown in Figure 46-10I for a passenger car was used to develop the sight distance shown in Figure 46-10J.

If stopping sight distance has been provided continuously along the major road and if sight distance for stop control or yield control has been provided for each minor-road approach, sight distance will be adequate for a left turn from the major road.

However, at a three-leg intersection located on or near a horizontal curve or crest vertical curve on the major road, the availability of adequate sight distance for a left turn from the major road should be checked. The availability of sight distance for a left turn from a divided highway should be checked because of the possibility of a sight obstruction in the median.

At a 4-leg intersection on a divided highway, an opposing vehicle turning left can block a driver’s view of oncoming traffic. The designer should consider offsetting the opposing left-turn lanes and providing a left-turning driver with a better view of oncoming traffic.

46-10.05 Signal-Controlled Intersection

If a vehicle is allowed to turn right on red, or left from a one-way street onto a one-way street, after stopping, the minimum ISD requirement shown in Figure 46-10H will apply to a signalized intersection. If this criterion cannot be met, consideration should be given to prohibiting right-turn-on-red at the intersection. This determination will be based on a field investigation and will be determined as required for each intersection leg. Changing right-turn-on-red regulations at an intersection will require an official action by State or local officials.

If the signal is to be placed on two-way flashing operation (i.e., flashing yellow on the major-road approaches and flashing red on the minor-road approaches) during off-peak or nighttime conditions,
the appropriate departure sight triangle for stop control, both to the left and to the right, should be provided for the minor-road approaches (See Section 46-10.03).

46-10.06 Effect of Skew

Where two highways intersect at an angle of less than 60 deg, some of the factors for determination of intersection sight distance may need adjustment.

Each of the clear-sight triangles described above is applicable to an oblique-angle intersection. As shown in Figure 46-10K, the leg of the sight triangle will lie along the intersection approach, and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of potential sight obstructions.

At an oblique-angle intersection, the length of the travel path for some turning and crossing maneuvers will be increased. The actual path length for a turning or crossing maneuver may be computed by dividing the total of the widths of the lanes (plus the median width, where appropriate) to be crossed, by the sine of the intersection angle. If the actual path length exceeds the total of widths of the lanes to be crossed by 12 ft or more, an appropriate number of additional lanes should be considered in applying the adjustment for the number of lanes to be crossed (See Section 46-10.03). For a crossing maneuver from a minor road with yield control, the \( w \) term in the equation for the major-road leg of the sight triangle to accommodate the crossing maneuver should also be divided by the sine of the intersection angle to obtain the actual path length. In the obtuse-angle quadrant of an oblique-angle intersection, the angle between the approach leg and the sight line may be so small that a driver can look across the full sight triangle with only a minor head movement. However, in the acute-angle quadrant, a driver is often required to turn his or her head considerably to see across the entire clear-sight triangle. The sight distance criteria for an intersection with no control should therefore not be applied to an oblique-angle intersection. Sight distance at least equal to that for an intersection with stop control on the minor road should be provided, where practical.
46-11.0  DRIVE DESIGN

46-11.01  General Information

46-11.01(01) Definitions of Drives and Types

The definitions of types and classes of drives are as follows:

1. Residential. A residential drive provides access to a single-family residence, duplex, or apartment building with not more than four dwelling units. A residential drive along a roadway with a raised curb is a class I drive. A residential drive along a roadway with a paved or unpaved shoulder and no raised curb is a class II drive.

2. Commercial. A commercial drive provides access to an office, retail, or institutional building, or to an apartment building with five or more dwelling units. A drive which serves an industrial plant, but with a primary function to serve an administrators’ or employees’ parking lot, is considered to be a commercial drive. A commercial drive along a roadway with a raised curb is a class III drive. A commercial drive along a roadway with a paved or unpaved shoulder and no raised curb is a class IV drive.

3. Industrial. An industrial drive directly serves substantial numbers of truck movements to and from loading docks of an industrial facility, warehouse, or truck terminal. A centralized retail development, such as a community or regional shopping center, may have one or more drives especially so designed, signed, and located to provide access for trucks. This is also classified as an industrial drive. An industrial drive may be designed either as a public road approach or as an industrial drive. An industrial drive along a roadway with a raised curb is a class VII drive. An industrial drive along a roadway with a paved or unpaved shoulder and no raised curb is a class VI drive.

4. Field Entrance. A field entrance provides access to an unimproved property, e.g., a farm field with no buildings. Such a drive along a roadway with a paved or unpaved shoulder is a class V drive.

46-11.01(02) Drive Spacing and Corner Clearance

Closely-spaced drives can cause operational problems, especially with a high-volume roadway or a high-volume drives. These problems can also result if a drive is too close to an at-grade intersection.
Any part of a drive, including its entrance radius, should not be placed within the radius of a public road at an intersection, including any auxiliary lanes. Preferably, there should be a 20- to 40-ft tangent section between the drive radius and the public-road radius for greater separation. If this criterion cannot be met for a property at an intersection corner, one solution may be to relocate the drive entrance from the major road to the minor road, if practical. Another possible solution is to provide a right-turn lane at the intersection. This will improve the operation of the intersection by removing the turning vehicles for the drive and intersection out of the through travel lane(s). However, a significant number of turning vehicles may impair egress from the property.

Drives for the same owner should be located across from each other (e.g., a farm) where crossing traffic is significant or where it is not desirable to permit slow or large equipment to travel along the highway or shoulder.

**46-11.01(03) Drive Sight Distance**

Section 46-10.0 discusses intersection sight distance (ISD) criteria for an intersection with a public road. These criteria will also apply to sight distance at a drive. However, for a drive with low traffic volume, it is not warranted to explore extraordinary measures to improve sight distance. Each sight obstruction, e.g., large tree, hedgerow, etc., should be checked for in the vicinity of the drive entrance which may limit sight distance. To perform the check, it is reasonable to assume an eye location of approximately 10 ft from the edge of travel lane.

If drive sight-distance criteria with the eye location described above cannot be met, informal notification should be provided to the project reviewer for a consultant-designed project or to the supervisor for an in-house project.

**46-11.01(04) Auxiliary Lane**

A deceleration or acceleration lane should be considered at each high-volume drive entrance, especially on a high-speed, high-volume arterial. Sections 46-4.0 and 46-7.0 further discuss the design and warrants for such an auxiliary lane, which may also apply to a high-volume drive. In addition to traffic-volume considerations, it may be warranted to provide a right-turn lane into the drive if the change in grade is abrupt at the drive entrance.

**46-11.01(05) Joint Residential or Commercial Drive**

If practical and agreeable to the property owners, the use of a joint drive offers one option to reduce...
the number of access points along the highway. The centerline of the joint drive should be located on
the property line dividing the two owners. This practice will not allow either owner the opportunity
to deny or restrict access to the neighbor’s property and, depending on the traffic volume, may
improve the traffic flow on the mainline. For a commercial drive, this may require providing a drive
wide enough to handle two-way traffic.

46-11.02 Design Criteria

The INDOT Standard Drawings provide the Department’s layout criteria for each drive class. In
addition, the following should be considered.

46-11.02(01) Class Determination Considerations

1. If it is determined from the survey or at the field inspection that a field entrance serves a barn
   or storage shed for farm machinery, it should be designed as a class II drive with a 24-ft
   minimum width instead of a class V drive.

2. Where there are positive indications that a private residence is being used for commercial
   purposes, the drive should be designed as a commercial drive.

46-11.02(02) Radii

1. Radii for a Class II or class IV drive should start from the edge of the paved shoulder if the
   width of the paved shoulder is 8 ft or greater.

2. Radii for a Class II or class IV drive should start from the edge of the traveled way if the width
   of the paved shoulder is less than 8 ft.

3. Class VI drive tapers should start from the edge of the traveled way without regard to the
   shoulder’s width or whether or not the shoulder is paved.

46-11.02(03) Width

1. Drive width should be measured perpendicular to the centerline of the drive.
2. For each new drive constructed where no drive currently exists, the minimum width shown on the INDOT Standard Drawings should be used, unless determined otherwise at the field inspection or if the Office of Real Estate recommends a wider drive.

3. The width of a reconstructed drive should be the same as the existing width but not less than the minimum width nor greater than the maximum width shown on the INDOT Standard Drawings.

4. Each drive that serves a barn or storage shed for farm equipment should be a minimum of 24 ft in width.

46-11.02(04) Drive Grade [Rev. Mar. 2016]

For a class I, III, VI, or VII drive, the maximum algebraic difference in drive grades should not exceed 8% for a crest vertical curve, or 12% for a sag vertical curve. For a class II, IV, or V drive, the maximum algebraic difference in drive grades should not exceed 11% for a crest vertical curve, or 14% for a sag vertical curve.

If it is known that a large emergency vehicle or other large vehicle will be using a drive, or if the algebraic differences exceed those noted above, the fit of the drive grade should be checked against the vehicle templates.

Drive grades should be shown and drive PVIs should be identified on the cross-sections sheets.

Where a drive is intersected by a sidewalk, the maximum drive grade is 2% for the width of the sidewalk. The preferred grade is 1.50% and should be used as a design practice to reduce the likelihood of the maximum being exceeded during construction. See Section 46-11.02(07) for additional information.

46-11.02(05) Grading

A drive’s embankment slope within the mainline clear zone should be as shown in Figure 46-11A, Drive Embankment Slope Within Clear Zone. Outside the clear zone, the embankment slope should be 4:1, but should not be steeper than 3:1.

46-11.02(06) Paving
1. Each residential, commercial, or industrial drive should have either an asphalt or concrete surface as shown on the INDOT Standard Drawings from the edge of the mainline pavement to at least the highway right-of-way line. The drive pavement should be replaced in kind beyond the right-of-way line only if required to match grade or alignment, and not to repair the drive due to condition.

2. A field entrance typically has an unimproved soil surface within the right-of-way, except as discussed in Section 46-11.02(01) Item 1.

46-11.02(07) Sidewalk-Driveway Crossings [Rev. Mar. 2016]

1. Where a sidewalk intersects a commercial drive that contains stop or yield control, a curb ramp should be used.

2. Where a sidewalk intersects a residential drive or commercial drive that does not contain stop or yield control, a sidewalk transition should be used.

In general, the difference between a sidewalk transition and a curb ramp is the need for a detectable warning surface. INDOT Standard Drawings Series E 604-SWCR and 604-SWDK contain curb ramp details and sidewalk-driveway crossing details, respectively.

46-11.03 Impacts to Project with Drive Design Complete and Right of Way Acquisition Under Way

Each Class I or III drive should have its grade designed in accordance with the INDOT Standard Drawings. However, if the profile-grade requirements shown on the Standard Drawings extend an already-designed drive outside the available right of way, such drive should have its grade detailed on the plans so that the drive remains inside the available right of way. Such drive should also be checked for accessibility by a large emergency vehicle or other large vehicle. Such drive should be identified as modified.

46-12.0 TURNING TEMPLATES

Figures 46-12A through 46-12P provide turning templates for the design vehicles most commonly used by INDOT. At least one turning template is included for each design vehicle listed below.

1. P Passenger car, light panel truck, or pickup truck
2. SU Single-unit truck
3. S-BUS-36 Conventional school bus (65 passengers)
4. WB-40 Intermediate semitrailer combination
5. WB-50 Intermediate semitrailer combination
6. WB-62 Interstate semitrailer combination
7. WB-65 (IDV) Indiana Design Vehicle: Interstate semitrailer combination
8. WB-109D Turnpike semitrailer combination with two trailers
9. MH/B Recreational vehicle: motor home and boat trailer
The list of the figures is as follows:

- **46-12A** P - Design Vehicle (1” = 20’ Scale)
- **46-12B** P - Design Vehicle (1” = 50’ Scale)
- **46-12C** SU - Design Vehicle (1” = 20’ Scale)
- **46-12D** SU - Design Vehicle (1” = 50’ Scale)
- **46-12D(1)** S-BUS-36 - Design Vehicle (1” = 20’ Scale)
- **46-12D(2)** S-BUS-36 - Design Vehicle (1” = 50’ Scale)
- **46-12E** WB-40 - Design Vehicle (1” = 30’ Scale)
- **46-12F** WB-40 - Design Vehicle (1” = 50’ Scale)
- **46-12G** WB-50 - Design Vehicle (1” = 30’ Scale)
- **46-12H** WB-50 - Design Vehicle (1” = 50’ Scale)
- **46-12I** WB-62 - Design Vehicle (1” = 30’ Scale)
- **46-12J** WB-62 - Design Vehicle (1” = 50’ Scale)
- **46-12K** WB-65 (IDV) - Design Vehicle (1” = 30’ Scale)
- **46-12L** WB-65 (IDV) - Design Vehicle (1” = 50’ Scale)
- **46-12N** WB-109D - Design Vehicle (1” = 50’ Scale)
- **46-12O** MH/B - Design Vehicle (1” = 30’ Scale)
- **46-12P** MH/B - Design Vehicle (1” = 50’ Scale)

Each figure shows the turning path for an above-listed AASHTO design vehicles. The path shown is for the left-front overhang and outside-rear wheel. The left-front wheel follows the circular curve; however, its path is not shown.
TREATMENTS FOR SKEWED INTERSECTIONS

Figure 46-1A
PROFILE AND CROSS-SECTION OF MAJOR HIGHWAY IS TYPICALLY MAINTAINED THROUGH AN INTERSECTION. IF BOTH INTERSECTING ROADS HAVE APPROXIMATELY EQUAL IMPORTANCE, BOTH ROADWAYS MAY BE TRANSITIONED.

MAJOR HIGHWAY

L_A (DES)

L_A (MIN)

TRANSITION MINOR ROAD SECTION TO MEET THE LONGITUDINAL GRADIENT OF THE MAJOR ROAD

MINOR HIGHWAY

L_B (MIN)

L_B DESIRABLE

LAST POINT OF TYPICAL CROSS SECTION

RADIUS RETURN

DES

PAVEMENT TRANSITIONS THROUGH INTERSECTIONS

L_A = TRANSITION LENGTH FOR MAJOR HIGHWAY

L_B = TRANSITION LENGTH FOR MINOR HIGHWAY

Figure 46-1B
MAXIMUM CHANGE IN GRADES WITHOUT A VERTICAL CURVE

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Crest Vertical Curve</th>
<th>Sag Vertical Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>ΔG = 7.0%</td>
<td>ΔG = 4.5%</td>
</tr>
<tr>
<td>25</td>
<td>ΔG = 5.0%</td>
<td>ΔG = 2.5%</td>
</tr>
<tr>
<td>30</td>
<td>ΔG = 3.0%</td>
<td>ΔG = 1.5%</td>
</tr>
</tbody>
</table>

Notes:

1. At a signalized intersection, the most desirable rotation option will be to transition all approach legs into a plane section through the intersection.

2. The grade of the approach roadway where vehicles may be stored should not be steeper than 1%. A grade steeper than 3% should be avoided.

3. The minor-road profile should tie into the major road’s travel lane cross slope as shown in the diagram. However, it will be acceptable for the minor-road profile to tie into the major road’s shoulder cross slope. Actual field conditions will determine the final design.

VERTICAL PROFILES OF INTERSECTING ROADS

Figure 46-1C
Note: If one of these standard public road approach types cannot be used at a particular intersection site, the public-road approach should be designed and the intersection details should be shown on the plans.

<table>
<thead>
<tr>
<th>Public-Road Approach</th>
<th>Appropriate Design Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type A</strong></td>
<td></td>
</tr>
<tr>
<td>● Paved- or unpaved-shoulder width &lt; 8 ft.</td>
<td>WB-50 or smaller</td>
</tr>
<tr>
<td>● Approach radius starts from edge of travel lane.</td>
<td></td>
</tr>
<tr>
<td>● Right-turn lane along mainline is not warranted.</td>
<td></td>
</tr>
<tr>
<td>● Serves residential, light-commercial, or light-industrial area.</td>
<td></td>
</tr>
<tr>
<td><strong>Type B</strong></td>
<td></td>
</tr>
<tr>
<td>● Paved-shoulder width ≥ 8 ft.</td>
<td>WB-50 or smaller</td>
</tr>
<tr>
<td>● Approach radius starts from edge of shoulder.</td>
<td></td>
</tr>
<tr>
<td>● Right-turn lane along mainline is not warranted.</td>
<td></td>
</tr>
<tr>
<td>● Serves residential, light-commercial, or light-industrial area.</td>
<td></td>
</tr>
<tr>
<td><strong>Type C</strong></td>
<td></td>
</tr>
<tr>
<td>● Paved-shoulder width ≥ 8 ft.</td>
<td>WB-50 or smaller</td>
</tr>
<tr>
<td>● Approach radius starts from edge of shoulder.</td>
<td></td>
</tr>
<tr>
<td>● Auxiliary right-turn lane along mainline is warranted.</td>
<td>WB-65 if adjoining traffic lanes are utilized.</td>
</tr>
<tr>
<td>● Serves residential, light-commercial, or light-industrial area.</td>
<td></td>
</tr>
<tr>
<td><strong>Type D</strong></td>
<td></td>
</tr>
<tr>
<td>● Paved-shoulder width ≥ 8 ft.</td>
<td>WB-65 or smaller</td>
</tr>
<tr>
<td>● Approach radius starts from edge of shoulder.</td>
<td></td>
</tr>
<tr>
<td>● Auxiliary right-turn lane along mainline is warranted.</td>
<td></td>
</tr>
<tr>
<td>● Used at intersection of two Department-maintained routes.</td>
<td></td>
</tr>
<tr>
<td>● Serves commercial area, heavy-industrial area, or truck stop.</td>
<td></td>
</tr>
</tbody>
</table>

PUBLIC-ROAD APPROACH TYPE
AND CORRESPONDING DESIGN VEHICLE

Figure 46-1C(1)
<table>
<thead>
<tr>
<th>For Turn Made From</th>
<th>For Turn Made Onto</th>
<th>Suggested Design Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Desirable</td>
</tr>
<tr>
<td>Freeway Ramp</td>
<td>Other Facility</td>
<td>IDV</td>
</tr>
<tr>
<td>Other Facility</td>
<td>Freeway Ramp</td>
<td>IDV</td>
</tr>
<tr>
<td>Arterial</td>
<td>Arterial Collector Local</td>
<td>IDV</td>
</tr>
<tr>
<td>Arterial</td>
<td>Collector Local</td>
<td>IDV</td>
</tr>
<tr>
<td>Collector</td>
<td>Arterial Collector Local</td>
<td>IDV</td>
</tr>
<tr>
<td>Collector</td>
<td>Collector Local</td>
<td>WB-50</td>
</tr>
<tr>
<td>Local</td>
<td>Arterial Collector Local</td>
<td>IDV</td>
</tr>
<tr>
<td>Local</td>
<td>Collector Local</td>
<td>SU*</td>
</tr>
</tbody>
</table>

*WB-50 can physically make the turn.*

**SUGGESTED DESIGN-VEHICLE SELECTION (Intersection)**

**Figure 46-1E**
<table>
<thead>
<tr>
<th>Turn Made From</th>
<th>Turn Made Onto</th>
<th>Acceptable Encroachment for Design Vehicle for Road or Street Onto Which Turn Made</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway Ramp</td>
<td>Other Facility</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
<tr>
<td>Arterial</td>
<td>Arterial</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
<tr>
<td>Collector</td>
<td>Collector</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
<tr>
<td>Local</td>
<td>Arterial</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
<tr>
<td>Local</td>
<td>Collector</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
<tr>
<td>Local</td>
<td>Local</td>
<td>No encroachment into opposing lanes of travel</td>
</tr>
</tbody>
</table>

Notes:

1. See Figure 46-1E for the design-vehicle selection. The encroachment criteria refer to the design vehicle.

2. Before the turn is made, the design vehicle is assumed to be in the outermost through travel lane or exclusive right-turn lane, whichever applies. It is assumed that the vehicle does not encroach onto adjacent lanes on the road or street from which the turn is made.

3. If determining the acceptable encroachment, the designer should also consider turning volume, through volume, and the type of traffic control at the intersection. The desirable encroachment will be zero into the opposing lanes of travel.

4. The table indicates the amount by which the turning vehicle can encroach into the opposing lanes of travel. If there are two or more lanes of traffic in the same direction on the road onto which the turn is made, the selected design vehicle can occupy both travel lanes. The turning vehicle will be able to make the turn while remaining entirely in the right through lane.

5. Regardless of the selected design vehicle or the criteria for encroachment, the IDV should physically be able to make a turn at an intersection without backing up and without impacting a curb, parked car, utility pole, mailbox, or other obstruction.

6. Each proposed design should be checked with the applicable vehicular turning template.

GUIDELINES FOR ENCROACHMENT FOR RIGHT TURN, URBAN INTERSECTION

Figure 46-2A
EFFECT OF CURB RADII AND PARKING ON TURNING PATHS

Figure 46-2B
TURNING RADIUS DESIGN

Figure 46-2C
<table>
<thead>
<tr>
<th>Angle of Turn, deg</th>
<th>Design Vehicle</th>
<th>Curve Radius, ft</th>
<th>Curve Radius with Taper, ft</th>
<th>Taper, H:V</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>P</td>
<td>60</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>SU</td>
<td>100</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>WB-40</td>
<td>150</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>WB-50</td>
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</tr>
<tr>
<td></td>
<td>WB-62</td>
<td>360</td>
<td>220</td>
<td>3</td>
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<tr>
<td></td>
<td>WB-67</td>
<td>380</td>
<td>220</td>
<td>3</td>
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<tr>
<td></td>
<td>WB-100T</td>
<td>260</td>
<td>125</td>
<td>3</td>
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<tr>
<td></td>
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<td>475</td>
<td>250</td>
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<tr>
<td>45</td>
<td>P</td>
<td>50</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>SU</td>
<td>75</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>WB-40</td>
<td>120</td>
<td>--</td>
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</tr>
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<td>175</td>
<td>120</td>
<td>2</td>
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<td></td>
<td>WB-62</td>
<td>230</td>
<td>145</td>
<td>4</td>
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<td></td>
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<td>250</td>
<td>145</td>
<td>4</td>
</tr>
<tr>
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<td>200</td>
<td>115</td>
<td>2.5</td>
</tr>
<tr>
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<td>--</td>
<td>200</td>
<td>4.5</td>
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<td>P</td>
<td>40</td>
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<td>--</td>
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<td>SU</td>
<td>60</td>
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<tr>
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<td>--</td>
<td>115</td>
<td>2.9</td>
</tr>
</tbody>
</table>

**EDGE-OF-TRAVELED-WAY DESIGN FOR TURN AT INTERSECTION**

Figure 46-2D
TYPICAL TURNING ROADWAY
(Stop Controlled on Minor Road)

Figure 46-3A
<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>50</td>
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<tr>
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<td>20</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
</tbody>
</table>

TAN. = Tangent.

**One-Lane, One-Way Operation, No Provision For Passing A Stall Vehicle**

**DERIVED PAVEMENT WIDTH, ft, FOR TURNING ROADWAY FOR EACH DESIGN VEHICLE**

Figure 46-3B
<table>
<thead>
<tr>
<th>Radius (ft)</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40 &amp; 45</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>2%-6%</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>100</td>
<td>2%-6%</td>
<td>2%-6%</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>150</td>
<td>2%-5%</td>
<td>2%-6%</td>
<td>4%-6%</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>225</td>
<td>2%-4%</td>
<td>2%-6%</td>
<td>3%-6%</td>
<td>6%</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>300</td>
<td>2%-3%</td>
<td>2%-4%</td>
<td>3%-6%</td>
<td>5%-6%</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>450</td>
<td>2%-3%</td>
<td>2%-3%</td>
<td>3%-5%</td>
<td>4%-6%</td>
<td>6%</td>
<td>—</td>
</tr>
<tr>
<td>600</td>
<td>2%</td>
<td>2%-3%</td>
<td>2%-4%</td>
<td>3%-5%</td>
<td>5%-6%</td>
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<td>4%-5%</td>
<td>5%-6%</td>
</tr>
<tr>
<td>1500</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
<td>2%-3%</td>
<td>3%-4%</td>
<td>4%-5%</td>
</tr>
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</table>

**SUPERELEVATION RATE FOR TURNING ROADWAY**

Figure 46-3C
DEVELOPMENT OF SUPERELEVATION AT
TURNING ROADWAY TERMINALS

Figure 46-3D
<table>
<thead>
<tr>
<th>Angle of Turn, deg</th>
<th>Design Classification</th>
<th>3-Centered Compound Curve Radii, ft</th>
<th>3-Centered Compound Curve Offset, ft</th>
<th>Lane Width, ft</th>
<th>Approx. Island Size, ft²</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>A</td>
<td>150-75-150</td>
<td>3.5</td>
<td>14</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>150-75-150</td>
<td>5.0</td>
<td>18</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>180-90-180</td>
<td>3.5</td>
<td>20</td>
<td>50</td>
</tr>
<tr>
<td>90</td>
<td>A</td>
<td>150-50-150</td>
<td>3.0</td>
<td>14</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>150-50-150</td>
<td>5.0</td>
<td>18</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>180-65-180</td>
<td>6.0</td>
<td>20</td>
<td>125</td>
</tr>
<tr>
<td>105</td>
<td>A</td>
<td>120-40-120</td>
<td>2.0</td>
<td>15</td>
<td>70</td>
</tr>
<tr>
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<td>B</td>
<td>100-35-100</td>
<td>5.0</td>
<td>22</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>180-45-180</td>
<td>8.0</td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>120</td>
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<td>100-30-100</td>
<td>2.5</td>
<td>16</td>
<td>120</td>
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<tr>
<td></td>
<td>B</td>
<td>100-30-100</td>
<td>5.0</td>
<td>24</td>
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<td>180-40-180</td>
<td>8.5</td>
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<td>16</td>
<td>460</td>
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<td>5.0</td>
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<td>370</td>
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<tr>
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<td>C</td>
<td>160-35-160</td>
<td>9.0</td>
<td>35</td>
<td>640</td>
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<tr>
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<td>A</td>
<td>100-30-100</td>
<td>2.5</td>
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<td>1400</td>
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<td>B</td>
<td>100-30-100</td>
<td>6.0</td>
<td>30</td>
<td>1170</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>160-35-160</td>
<td>7.1</td>
<td>38</td>
<td>1720</td>
</tr>
</tbody>
</table>

1. An asymmetric three-centered compound curve or straight tapers with a simple curve may also be used without significantly altering the roadway width or the corner-island size. Painted island delineation is recommended for an island of less than 75 ft² in area.

2. Design classification is defined as follows:
   A: Primarily P. Permits occasional design single-unit truck to turn with restricted clearances.
   B: Provides adequately for SU. Permits occasional WB-50 to turn with slight encroachment onto adjacent traffic lanes.
   C: Provides fully for WB-50.

**DESIGN FOR TURNING ROADWAY**

*Figure 46-3E*
Design Speed (mph) | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45
Stopping Sight Distance (ft) | 50 | 80 | 115 | 155 | 200 | 250 | 305 | 360

STOPPING SIGHT DISTANCE FOR TURNING ROADWAY

Figure 46-3E(1)
<table>
<thead>
<tr>
<th>Design Speed of Curve at Section D-D* (mph)</th>
<th>Maximum Algebraic Difference in Cross Slope at Crossover Line (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15 to 20</td>
<td>5 to 8</td>
</tr>
<tr>
<td>25 to 30</td>
<td>5 to 6</td>
</tr>
<tr>
<td>&gt; 30</td>
<td>4 to 5</td>
</tr>
</tbody>
</table>

* See Figure 46-3D

Note: Figure is also applicable to a turn lane or where shoulder is anticipated to be used as a turn lane.

PAVEMENT CROSS SLOPE AT TURNING-ROADWAY TERMINAL

Figure 46-3F
ADDITIONAL LENGTH OF TURNING ROADWAY
(Signalized Intersection)

Figure 46-3G
TYPICAL PAVEMENT MARKINGS FOR TURNING ROADWAYS

Figure 46-3H
GUIDELINES FOR RIGHT-TURN LANES AT UNSIGNALIZED INTERSECTIONS ON 2-LANE HIGHWAYS

Figure 46-4A
GUIDELINES FOR RIGHT-TURN LANE S AT UNSIGNALIZED INTERSECTION ON 4-LANE HIGHWAYS

Figure 46-4B
<table>
<thead>
<tr>
<th>Operating Speed (mph)</th>
<th>Opposing Volume (veh/h)</th>
<th>5% Left Turns</th>
<th>10% Left Turns</th>
<th>20% Left Turns</th>
<th>30% Left Turns</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>800</td>
<td>330</td>
<td>240</td>
<td>180</td>
<td>160</td>
</tr>
<tr>
<td></td>
<td>600</td>
<td>410</td>
<td>305</td>
<td>225</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>510</td>
<td>380</td>
<td>275</td>
<td>245</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>640</td>
<td>470</td>
<td>350</td>
<td>305</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>720</td>
<td>515</td>
<td>390</td>
<td>340</td>
</tr>
<tr>
<td>50</td>
<td>800</td>
<td>280</td>
<td>210</td>
<td>165</td>
<td>135</td>
</tr>
<tr>
<td></td>
<td>600</td>
<td>350</td>
<td>260</td>
<td>195</td>
<td>170</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>430</td>
<td>320</td>
<td>240</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>550</td>
<td>400</td>
<td>300</td>
<td>270</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>615</td>
<td>445</td>
<td>335</td>
<td>295</td>
</tr>
<tr>
<td>60</td>
<td>800</td>
<td>230</td>
<td>170</td>
<td>125</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>600</td>
<td>290</td>
<td>210</td>
<td>160</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>365</td>
<td>270</td>
<td>200</td>
<td>175</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>450</td>
<td>330</td>
<td>250</td>
<td>215</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>505</td>
<td>370</td>
<td>275</td>
<td>240</td>
</tr>
</tbody>
</table>

**VOLUME GUIDELINES FOR LEFT-TURN LANE ON TWO-LANE HIGHWAY**

*Figure 46-4C*
<table>
<thead>
<tr>
<th>Classification</th>
<th>Functional Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Arterial</td>
<td>$L_T + L_D + L_S$</td>
</tr>
<tr>
<td>Urban Arterial Other Facility</td>
<td>$L_T + L_D + L_S$  (Desirable)</td>
</tr>
<tr>
<td>Stop or T Facility</td>
<td>$L_T + L_S$       (Minimum)</td>
</tr>
</tbody>
</table>

$L_T = \text{Length of Taper (100 ft or more)}$

$L_D = \text{Length of Deceleration}$

$L_S = \text{Length of Storage}$

Notes:

1. See Figure 46-4I for an illustration of the terms.

2. $L_D$ is a consideration only at a free-flowing leg of a stop-controlled intersection or signalized intersection, or at a free-flowing turning roadway with a turn lane.

FUNCTIONAL LENGTH OF AUXILIARY TURN LANE

Figure 46-4H
The schematic of the major road (free flowing) also applies to all legs of a signalized intersection.

NOTE: The schematic of the major road (free flowing) also applies to all legs of a signalized intersection.

KEY: $L_T = \text{Taper Length (100' or more)}$
$L_D = \text{Deceleration Length}$
$L_S = \text{Storage Length}$

TYPICAL AUXILIARY LANES AT AN INTERSECTION

Figure 46-4 I
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>$L_{D_s}$ Full-Width Auxiliary Lane (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>530</td>
</tr>
<tr>
<td>55</td>
<td>480</td>
</tr>
<tr>
<td>50</td>
<td>435</td>
</tr>
<tr>
<td>45</td>
<td>385</td>
</tr>
<tr>
<td>40</td>
<td>320</td>
</tr>
<tr>
<td>35</td>
<td>280</td>
</tr>
<tr>
<td>30</td>
<td>235</td>
</tr>
<tr>
<td>25</td>
<td>200</td>
</tr>
</tbody>
</table>

Grade-Adjustment Factor for Downgrade, $G_d$

<table>
<thead>
<tr>
<th>$0 \leq G_d &lt; 2$</th>
<th>$2 \leq G_d &lt; 3$</th>
<th>$3 \leq G_d &lt; 4$</th>
<th>$4 \leq G_d &lt; 5$</th>
<th>$5 \leq G_d \leq 6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>1.10</td>
<td>1.20</td>
<td>1.28</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Grade-Adjustment Factor for Upgrade, $G_u$

<table>
<thead>
<tr>
<th>$0 \leq G_u &lt; 2$</th>
<th>$2 \leq G_u &lt; 3$</th>
<th>$3 \leq G_u &lt; 4$</th>
<th>$4 \leq G_u &lt; 5$</th>
<th>$5 \leq G_u \leq 6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Note: The grade-adjustment factor multiplied by the length $L_D$ provided above will provide the deceleration-lane length adjusted for grade. The adjustment factor applies to each design speed.
RECOMMENDED STORAGE LENGTH FOR SIGNALIZED INTERSECTIONS

Figure 46-4K
<table>
<thead>
<tr>
<th>$v/c$ RATIO, $X$</th>
<th>CYCLE LENGTH, $C$ (s)</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>0.70</td>
<td>0.76</td>
<td>0.84</td>
<td>0.89</td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>0.55</td>
<td>0.71</td>
<td>0.77</td>
<td>0.85</td>
<td>0.90</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>0.60</td>
<td>0.73</td>
<td>0.79</td>
<td>0.87</td>
<td>0.92</td>
<td>0.97</td>
<td></td>
</tr>
<tr>
<td>0.65</td>
<td>0.75</td>
<td>0.81</td>
<td>0.89</td>
<td>0.94</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>0.70</td>
<td>0.77</td>
<td>0.84</td>
<td>0.92</td>
<td>0.98</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>0.82</td>
<td>0.88</td>
<td>0.98</td>
<td>1.03</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>0.80</td>
<td>0.88</td>
<td>0.95</td>
<td>1.05</td>
<td>1.11</td>
<td>1.17</td>
<td></td>
</tr>
<tr>
<td>0.85</td>
<td>0.99</td>
<td>1.06</td>
<td>1.18</td>
<td>1.24</td>
<td>1.31</td>
<td></td>
</tr>
<tr>
<td>0.90</td>
<td>1.17</td>
<td>1.26</td>
<td>1.40</td>
<td>1.48</td>
<td>1.56</td>
<td></td>
</tr>
<tr>
<td>0.95</td>
<td>1.61</td>
<td>1.74</td>
<td>1.92</td>
<td>2.03</td>
<td>2.14</td>
<td></td>
</tr>
</tbody>
</table>

Notes:

1. **Table applies to an exclusive left-turn lane or an exclusive right-turn lane where a turn on red is not permitted.**

2. **See minimum storage length discussion in Section 46-4.02(02).**

3. **To determine the $v/c$ ratio and the passenger car equivalent (PCE) values, see the Highway Capacity Manual.**

**RECOMMENDED STORAGE LENGTH FOR SIGNALIZED INTERSECTION**

*Figure 46-4K(1)*
<table>
<thead>
<tr>
<th>TURNING DHV (vph)</th>
<th>$L_s$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 60$</td>
<td>50 to 75</td>
</tr>
<tr>
<td>$60 &lt; DHV \leq 120$</td>
<td>100</td>
</tr>
<tr>
<td>$120 &lt; DHV \leq 180$</td>
<td>150</td>
</tr>
<tr>
<td>$&gt; 180$</td>
<td>$\geq 200$</td>
</tr>
</tbody>
</table>

Note: See Section 46-4.02(02) for minimum storage-length criteria.

RECOMMENDED STORAGE LENGTH, $L_s$, FOR UNSIGNALIZED INTERSECTION

Figure 46-4L
<table>
<thead>
<tr>
<th>Design Speed, $S$ (mph)</th>
<th>Taper Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>10:1</td>
</tr>
<tr>
<td>25</td>
<td>10:1</td>
</tr>
<tr>
<td>30</td>
<td>15:1</td>
</tr>
<tr>
<td>35</td>
<td>20:1</td>
</tr>
<tr>
<td>40</td>
<td>30:1</td>
</tr>
<tr>
<td>45</td>
<td>45:1</td>
</tr>
<tr>
<td>50</td>
<td>50:1</td>
</tr>
<tr>
<td>55</td>
<td>55:1</td>
</tr>
<tr>
<td>60</td>
<td>60:1</td>
</tr>
</tbody>
</table>

Taper Rate = $S$ for $S \geq 45$ mph

Taper Rate = $\frac{S^2}{60}$ for $S < 45$ mph

$W =$ Horizontal lane shift, ft

$L =$ $W \times S$ ($S \geq 45$ mph)

$L =$ $W \times \frac{S^2}{60}$ ($S < 45$ mph)

*See Section 46-4.02 for minimum turn-lane length.

CHANNELIZED TURN LANE FOR 2-LANE HIGHWAY

Figure 46-4M
Notes:

1. In rural areas, a minimum length of 100' is required. In urban areas, low-speed conditions, 50' is acceptable.

2. Minimum island width at stop line is 6'.

3. Where a pedestrian refuge is provided, the minimum width is 6'.

4. This figure represents the minimum conditions for slotted tapered left-turn lane construction. If minimums cannot be met, consideration should be given to other solutions, e.g., left-turn prohibition, split phasing.

5. See Section 502-2.0 for pavement markings.

**Typical Slotted Tapered Left-Turn Lane**

(Signalized Intersection)

Figure 46-4N
NOTES:

1. In rural areas, a minimum length of 100' is required. In urban areas, low-speed conditions, 50' is acceptable.
2. Minimum island width at stop line is 6'.
3. Where a pedestrian refuge is provided, the minimum width is 6'.
4. This figure represents the minimum conditions for slotted parallel left-turn lane construction. If minimums cannot be met, consideration should be given to other solutions, e.g., left-turn prohibition, split phasing.
5. See Section 502-2.0 for pavement markings.

TYPICAL SLOTTED PARALLEL LEFT-TURN LANE
(Signalized Intersection)

Figure 46-4N(1)
### Minimum Dimensions for Passing Blister

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>$T_1$ (ft)</th>
<th>$L$ (ft)</th>
<th>$T_2$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 or less</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Greater than 30, but less than 50</td>
<td>200</td>
<td>150</td>
<td>200</td>
</tr>
<tr>
<td>50 or greater</td>
<td>300</td>
<td>200</td>
<td>300</td>
</tr>
</tbody>
</table>

**Note:** For shoulder widths adjacent to the passing blister, see auxiliary widths in Chapters Fifty-three and Fifty-five.

**Typical Passing Blister for a 2-Lane Highway**

*Figure 46-40*
CONSIDER PROVIDING SPECIAL PAVEMENT MARKINGS (GUIDE LINES) TO HELP GUIDE VEHICLES TURNING FROM MULTIPLE TURN LANES

ADJUST THROAT WIDTH TO ACCOMMODATE MULTIPLE LEFT-TURN LANES

THIS DIMENSION APPLIES TO THE SEPARATION OF OPPOSING MULTIPLE LEFT-TURN LANES TURNING SIMULTANEOUSLY

ADJUST THROAT WIDTH TO ACCOMMODATE MULTIPLE LEFT AND RIGHT-TURN LANES

SCHEMATIC FOR MULTIPLE TURN LANES

Figure 46-4P
Note: See Chapter Seventy-six for additional information on pavement marking details.

TYPICAL PAVEMENT MARKINGS FOR A TWLTL

Figure 46-5A
**NOTES:**

1. $D_E$ is that distance required by the vehicle to accelerate from a stop to 6 mph below the average running speed.

2. If driveways are present, $D_E$ may need to be increased.

3. Sharper taper rates may be used in urban areas, however the taper lengths should be at least 300 ft.

4. These criteria are for preliminary design purposes. The final design will be determined on a case-by-case basis.

---

**EXTENSION OF ADDITIONAL THROUGH LANES**

**Figure 46-7A**
RECOMMENDED MEDIAN OPENING SPACING
(Non-Freeway)

Figure 46-8A
<table>
<thead>
<tr>
<th>Median Width, $M$ (ft)</th>
<th>Minimum Length of Median Opening, $L$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Semicircular</td>
</tr>
<tr>
<td>4</td>
<td>130</td>
</tr>
<tr>
<td>16</td>
<td>120</td>
</tr>
<tr>
<td>20</td>
<td>115</td>
</tr>
<tr>
<td>28</td>
<td>105</td>
</tr>
<tr>
<td>36</td>
<td>95</td>
</tr>
<tr>
<td>50</td>
<td>85</td>
</tr>
<tr>
<td>60</td>
<td>75</td>
</tr>
</tbody>
</table>

**MINIMUM DESIGN OF MEDIAN OPENING**
(Control Radius of 70 ft)

**Figure 46-8C**
<table>
<thead>
<tr>
<th>Median Width, ( M ) (ft)</th>
<th>Minimum Length of Median Opening, ( L ) (ft)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>165</td>
<td>150</td>
</tr>
<tr>
<td>16</td>
<td>150</td>
<td>105</td>
</tr>
<tr>
<td>20</td>
<td>145</td>
<td>100</td>
</tr>
<tr>
<td>28</td>
<td>140</td>
<td>80</td>
</tr>
<tr>
<td>36</td>
<td>130</td>
<td>70</td>
</tr>
<tr>
<td>50</td>
<td>115</td>
<td>55</td>
</tr>
<tr>
<td>60</td>
<td>105</td>
<td>45</td>
</tr>
</tbody>
</table>

**MINIMUM DESIGN OF MEDIAN OPENING**
*(Control Radius of 85 ft)*

Figure 46-8D
<table>
<thead>
<tr>
<th>Median Width, $M$ (ft)</th>
<th>Minimum Length of Median Opening, $L$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Semicircular</td>
</tr>
<tr>
<td>4</td>
<td>195</td>
</tr>
<tr>
<td>16</td>
<td>185</td>
</tr>
<tr>
<td>20</td>
<td>180</td>
</tr>
<tr>
<td>28</td>
<td>170</td>
</tr>
<tr>
<td>36</td>
<td>160</td>
</tr>
<tr>
<td>50</td>
<td>150</td>
</tr>
<tr>
<td>60</td>
<td>140</td>
</tr>
</tbody>
</table>

**MINIMUM DESIGN OF MEDIAN OPENING**

*(Control Radius of 100 ft)*

*Figure 46-8E*
TRIANGULAR ISLAND

Figure 46-9A
PAINTED FLUSH ISLAND

RAISED CORRUGATED ISLAND

ELONGATED ISLANDS

Figure 46-9B
EXAMPLE OF A CHANNELIZING INTERSECTION

Figure 46-9C
Example

Given: No traffic control at intersection
Design speed – 40 mph (Highway A)
25 km/h (Highway B)

Problem: Determine legs of sight triangle.
Solution: From above table —
$\text{ISD}_a = 180 \text{ ft}$
$\text{ISD}_b = 110 \text{ ft}$

Note: This figure is not applicable for State highways.

INTERSECTION SIGHT DISTANCE
(No traffic control)
Figure 46-10A
<table>
<thead>
<tr>
<th>Approach Grade (%)</th>
<th>Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>15</td>
</tr>
<tr>
<td>-6</td>
<td>1.1</td>
</tr>
<tr>
<td>-5</td>
<td>1.0</td>
</tr>
<tr>
<td>-4</td>
<td>1.0</td>
</tr>
<tr>
<td>-3 to +3</td>
<td>1.0</td>
</tr>
<tr>
<td>+4</td>
<td>1.0</td>
</tr>
<tr>
<td>+5</td>
<td>1.0</td>
</tr>
<tr>
<td>+6</td>
<td>1.0</td>
</tr>
</tbody>
</table>

*Note: Factor is based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.*

**ADJUSTMENT FACTOR FOR SIGHT DISTANCE WITH NO TRAFFIC CONTROL**

*Figure 46-10B*
<table>
<thead>
<tr>
<th>Design Vehicle</th>
<th>Time Gap, $t_g$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>8</td>
</tr>
<tr>
<td>Single-Unit Truck</td>
<td>10</td>
</tr>
<tr>
<td>Combination Truck</td>
<td>12</td>
</tr>
</tbody>
</table>

Note: Time gap is for a vehicle to turn right or left onto a two-lane highway with no median. The table values require adjustments for a multilane highway as follows:

1. For a left turn onto a two-way highway with more than two lanes, add 0.5 s for a passenger car or 0.7 s for a truck for each additional lane, from left, in excess of one, to be crossed by the turning vehicle.

2. For a right turn, no adjustment is necessary.

TIME GAP FOR LEFT OR RIGHT TURN, YIELD CONTROL

Figure 46-10C
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Length of Leg, Passenger Car (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>180</td>
</tr>
<tr>
<td>20</td>
<td>240</td>
</tr>
<tr>
<td>25</td>
<td>295</td>
</tr>
<tr>
<td>30</td>
<td>355</td>
</tr>
<tr>
<td>35</td>
<td>415</td>
</tr>
<tr>
<td>40</td>
<td>475</td>
</tr>
<tr>
<td>45</td>
<td>530</td>
</tr>
<tr>
<td>50</td>
<td>590</td>
</tr>
<tr>
<td>55</td>
<td>650</td>
</tr>
</tbody>
</table>

Note: Distance shown is for passenger car making right or left turn onto two-lane road without stopping.

DESIGN INTERSECTION SIGHT DISTANCE, LEFT OR RIGHT TURN AT YIELD-CONTROLLED INTERSECTION

Figure 46-10D
INTERSECTION SIGHT DISTANCE FOR TURNING ROADWAYS

Figure 46-10E
Clear Sight Triangle for Viewing Traffic Approaching from the Left.

Clear Sight Triangle for Viewing Traffic Approaching from the Right.

DEPARTURE SIGHT TRIANGLES

Figure 46-10F
<table>
<thead>
<tr>
<th>$V_{\text{major}}$ (mph)</th>
<th>Passenger Car</th>
<th></th>
<th>Single-Unit Truck</th>
<th>Combination Truck</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Local Road</td>
<td>Collector or Arterial</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$t_g$ (s)</td>
<td>ISD (ft)</td>
<td>$t_g$ (s)</td>
<td>ISD (ft)</td>
</tr>
<tr>
<td>15</td>
<td>7.5</td>
<td>170</td>
<td>7.5</td>
<td>170</td>
</tr>
<tr>
<td>20</td>
<td>7.5</td>
<td>220</td>
<td>7.5</td>
<td>220</td>
</tr>
<tr>
<td>25</td>
<td>7.5</td>
<td>280</td>
<td>7.5</td>
<td>280</td>
</tr>
<tr>
<td>30</td>
<td>7.5</td>
<td>330</td>
<td>7.5</td>
<td>330</td>
</tr>
<tr>
<td>35</td>
<td>7.5</td>
<td>390</td>
<td>7.5</td>
<td>390</td>
</tr>
<tr>
<td>40</td>
<td>7.5</td>
<td>440</td>
<td>7.5</td>
<td>440</td>
</tr>
<tr>
<td>45</td>
<td>7.5</td>
<td>500</td>
<td>7.5</td>
<td>500</td>
</tr>
<tr>
<td>50</td>
<td>7.5</td>
<td>550</td>
<td>8.5</td>
<td>630</td>
</tr>
<tr>
<td>55</td>
<td>7.5</td>
<td>610</td>
<td>9.0</td>
<td>730</td>
</tr>
<tr>
<td>60</td>
<td>7.5</td>
<td>670</td>
<td>9.5</td>
<td>840</td>
</tr>
<tr>
<td>65</td>
<td>7.5</td>
<td>720</td>
<td>10.0</td>
<td>960</td>
</tr>
<tr>
<td>70</td>
<td>7.5</td>
<td>780</td>
<td>10.0</td>
<td>1030</td>
</tr>
</tbody>
</table>

$V_{\text{major}}$ = Design speed of major road

$t_g$ = Time gap for minor road vehicle to enter major road

ISD = Intersection sight distance (length of leg of sight triangle along major road)

ISD is shown for a stopped vehicle to turn left onto a two-lane highway with approach grades of 3% or flatter. For other conditions, the time gap should be adjusted and the required ISD recalculated using the formula ISD = 1.47 $V_{\text{major}}$ $t_g$.

For a left turn onto a two-way highway with more than two lanes, add 0.5 s for a passenger car, or 0.7 s for a truck for each additional lane from the left in excess of one, to be crossed by a turning vehicle.

For the minor-road approach, if its grade is an upgrade that is steeper than 3%, add 0.2 s for each percent grade for a left turn. The adjustment for the minor-road approach grade is required only if the rear wheels of the design vehicle would be on an upgrade steeper than 3%.

INTERSECTION SIGHT DISTANCE FOR STOP-CONTROLLED INTERSECTION

Figure 46-10G
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Intersection Sight Distance For Passenger Car Calculated (ft)</th>
<th>Design (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>143.3</td>
<td>145</td>
</tr>
<tr>
<td>20</td>
<td>191.1</td>
<td>195</td>
</tr>
<tr>
<td>25</td>
<td>238.9</td>
<td>240</td>
</tr>
<tr>
<td>30</td>
<td>286.7</td>
<td>290</td>
</tr>
<tr>
<td>35</td>
<td>334.4</td>
<td>335</td>
</tr>
<tr>
<td>40</td>
<td>382.2</td>
<td>385</td>
</tr>
<tr>
<td>45</td>
<td>430.0</td>
<td>430</td>
</tr>
<tr>
<td>50</td>
<td>477.8</td>
<td>480</td>
</tr>
<tr>
<td>55</td>
<td>525.5</td>
<td>530</td>
</tr>
<tr>
<td>60</td>
<td>573.3</td>
<td>575</td>
</tr>
<tr>
<td>65</td>
<td>621.1</td>
<td>625</td>
</tr>
<tr>
<td>70</td>
<td>668.9</td>
<td>670</td>
</tr>
</tbody>
</table>

Note: Intersection sight distance shown is for a stopped passenger car to turn right onto or cross a two-lane highway with no median and grades of 3% or flatter. For other conditions, the time gap should be adjusted and the required sight distance recalculated.

INTERSECTION SIGHT DISTANCE FOR PASSENGER CAR TO TURN RIGHT FROM A STOP OR TO MAKE A CROSSING MANEUVER

Figure 46-10H
**Design Vehicle Time Gap, $t_g$ (s)**

<table>
<thead>
<tr>
<th>Design Vehicle</th>
<th>Time Gap, $t_g$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>6.5</td>
</tr>
<tr>
<td>Single-Unit Truck</td>
<td>8.5</td>
</tr>
<tr>
<td>Combination Truck</td>
<td>10.5</td>
</tr>
</tbody>
</table>

* For crossing a major road with more than two lanes, add 0.5 s for a passenger car, or 0.7 s for a truck, for each additional lane or narrow median to be crossed. A narrow median is one that cannot store the design vehicle.

If the minor-road approach grade is an upgrade that is steeper than 3%, add 0.1 s for each percent grade.

**TIME GAP FOR CROSSING MANEUVER**

Figure 46-10H(1)
### Design Vehicle Time Gap, \( t_g \) (s)*

<table>
<thead>
<tr>
<th>Design Vehicle</th>
<th>Time Gap, ( t_g ) (s)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>5.5</td>
</tr>
<tr>
<td>Single-unit truck</td>
<td>6.5</td>
</tr>
<tr>
<td>Combination truck</td>
<td>7.5</td>
</tr>
</tbody>
</table>

**Divided Highway:** For a left-turning vehicle crossing more than one opposing lane, add 0.5 s for a passenger car, or 0.7 s for a truck, for each additional lane to be crossed and for a narrow median that cannot store the design vehicle.

**Minor Road Approach Grade:** If the approach grade is an upgrade that is steeper than 3%, add 0.1 s for each percent grade.

**TIME GAP FOR LEFT TURN FROM THE MAJOR ROAD**

*Figure 46-10*
## Intersection Sight Distance

### For Passenger Car

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Stopping Sight Distance (ft)</th>
<th>Calculated (ft)</th>
<th>Design (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>80</td>
<td>121.3</td>
<td>125</td>
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<tr>
<td>20</td>
<td>115</td>
<td>161.7</td>
<td>165</td>
</tr>
<tr>
<td>25</td>
<td>155</td>
<td>202.1</td>
<td>205</td>
</tr>
<tr>
<td>30</td>
<td>200</td>
<td>242.6</td>
<td>245</td>
</tr>
<tr>
<td>35</td>
<td>250</td>
<td>283.0</td>
<td>285</td>
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<tr>
<td>40</td>
<td>305</td>
<td>323.4</td>
<td>325</td>
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<td>45</td>
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<td>404.3</td>
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<td>55</td>
<td>495</td>
<td>444.7</td>
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<td>60</td>
<td>570</td>
<td>485.1</td>
<td>490</td>
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<tr>
<td>65</td>
<td>645</td>
<td>525.5</td>
<td>530</td>
</tr>
<tr>
<td>70</td>
<td>730</td>
<td>566.0</td>
<td>570</td>
</tr>
</tbody>
</table>

**Note:** ISD is shown for a passenger car making a left turn from an undivided highway. For other conditions and design vehicles, the time gap should be adjusted and the required ISD recalculated.

---

**INTERSECTION SIGHT DISTANCE**  
FOR LEFT TURN FROM MAJOR ROAD

**Figure 46-10J**
W_2 = \frac{W_1}{\sin \Theta}

SIGHT TRIANGLES AT SKEWED INTERSECTIONS

Figure 46-10K
<table>
<thead>
<tr>
<th>Slope</th>
<th>Divided Highway, ( \geq 4 ) Lanes</th>
<th>Other Arterial</th>
<th>Collector</th>
<th>Local Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>10:1</td>
<td>All</td>
<td>All</td>
<td>Design Speed ( \geq 50 ) mph</td>
<td>n/a</td>
</tr>
<tr>
<td>6:1</td>
<td>n/a</td>
<td>All</td>
<td>Design Speed ( \geq 50 ) mph</td>
<td>n/a</td>
</tr>
<tr>
<td>4:1</td>
<td>n/a</td>
<td>n/a</td>
<td>Design Speed ( \leq 45 ) mph</td>
<td>All</td>
</tr>
</tbody>
</table>

**DRIVE EMBANKMENT SLOPES WITHIN CLEAR ZONE**

*Figure 46-11A*
P - DESIGN VEHICLE
(1" = 20' SCALE)

Figure 46-12A
P - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46-12B
SU - DESIGN VEHICLE
(1" = 20' SCALE)

Figure 46-12C
SU - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46 -12D
MAX. STEERING ANGLE = 37.1 deg.

S-BUS-36 - DESIGN VEHICLE
(1" = 20' SCALE)

Figure 46 -12D(1)
S-BUS-36 - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46 -12D(2)
MAX. STEERING ANGLE = 20.4 deg.

OUTSIDE SWEPT PATH

INSIDE SWEPT PATH

WB-40 - DESIGN VEHICLE
(1" = 30' SCALE)

Figure 46 -12E
WB-40 - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46 -12F
MAX. STEERING ANGLE = 17.8 deg.
MAX. STEERING ANGLE = 17.8 deg.

OUTSIDE SWEPT PATH

INSIDE SWEPT PATH

WB-50 - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46 -12H
MAX. STEERING ANGLE = 28.4 deg.
MAX. STEERING ANGLE = 28.4 deg.
WB-65 (IDV) - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46-12L
WB-109D - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46-12N
MH/B - DESIGN VEHICLE
(1" = 30' SCALE)

Figure 46 -12 O
MH/B - DESIGN VEHICLE
(1" = 50' SCALE)

Figure 46 -12P
NOTE: This chapter is currently being re-written and its content will be included in Chapter 306 in the future.

CHAPTER 48

Interchanges

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<th>Sections Affected</th>
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<td>All sections revised.</td>
</tr>
<tr>
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CHAPTER 48

INTERCHANGES

48-1.0 GENERAL

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways on different levels.

48-1.01 INDOT Procedures

The Traffic Engineering Division Corridor Development Office is generally responsible for determining the need for, location of and type of interchanges. This assessment is based on a consideration of several factors, which are discussed in Sections 48-1.0 and 48-2.0. The designer is responsible for determining the layout and design of the interchange as discussed in Sections 48-3.0 through 48-6.0.

48-1.02 Guidelines

Although an interchange is a high-level compromise for intersection problems, its high cost and environmental impact require that an interchange be used only after careful consideration of its benefits. Because of the great variance in specific site conditions, INDOT has not adopted specific interchange warrants. Consider the following when determining the need for an interchange or grade separation:

1. **Design Designation.** Once it has been decided to provide a fully access-controlled facility, each intersecting highway must be terminated, rerouted, provided a grade separation or provided an interchange. The importance of the continuity of the crossing road and the feasibility of an alternative route will determine the need for a grade separation or interchange. An interchange should be provided on the basis of the anticipated demand for access to the minor road.

   On facilities with partial control of access, intersections with public roads will be accommodated by an interchange or with an at-grade intersection; grade separations alone are not normally provided. Typically, an interchange will be selected for the higher-volume intersecting roads. Therefore, on a facility with partial control of access, the decision to provide an interchange will be, in general, based on the criteria in Section 48-1.04.
2. **Congestion.** An interchange may be considered where the level of service (LOS) at an at-grade intersection is unacceptable, and the intersection cannot be redesigned at-grade to operate at an acceptable LOS. Although LOS criteria is the most tangible of any interchange guideline, INDOT has not adopted any specific levels which, when exceeded, would demand an interchange. Even on facilities with partial control of access, the elimination of signalization contributes greatly to the improvement of flow.

3. **Safety.** The accident reduction benefits of an interchange should be considered at an existing at-grade intersection which has a high accident rate. The elimination of railroad-highway crossings should be considered in this factor. Section 48-3.08 provides additional information on various safety considerations relative to interchange selection.

4. **Site Topography.** At some sites the topography may be more adaptable to an interchange than an at-grade intersection.

5. **Road-User Benefits.** Interchanges significantly reduce the travel time when compared to at-grade intersections but may increase the travel distances. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be considered. For more information on road-user benefit analysis, see Chapter 50.

6. **Traffic Volume.** Interchanges should be considered at crossroads with heavy traffic volumes because elimination of conflicts greatly improves the movement of traffic.

7. **Other Factors.** Other factors, which need to be considered, include construction costs, right-of-way impacts and environmental concerns.

---

### 48-1.03 New or Revised Access to the Interstate System

#### 48-1.03(01) Applicability

Each entrance or exit point to an Interstate freeway route is considered an access point. For example, a conventional diamond interchange has four access points, two on-ramps and two off-ramps. Locked-gate access is defined as an access point, and is described in Section 48-1.03(02), Item 9.

Revised access to an Interstate route is a change in the existing essential form, even though the sheer number of access points does not change. For example, adding a loop on-ramp in concert with a collector-distributor (C-D) roadway linked with a downstream diagonal on ramp to an otherwise conventional diamond interchange, or changing a cloverleaf interchange into a fully-
directional interchange is considered revised access. Lengthening or adding auxiliary lanes at at-grade ramp terminals with crossroads or ramp-proper lanes is not considering revised access, nor is converting a single-lane off- or on-ramp to dual-lanes. This is clarified in Sections 48-1.03(02) and 48-1.03(03).

The design of new or revised access must comply with AASHTO’s *A Policy on Geometric Design of Highways and Streets* (AASHTO GDHS), AASHTO’s *A Policy on Design Standards – Interstate System* (Interstate Standards), and this manual.

Work determined to consist of new or revised access to the existing Interstate System will require development by INDOT of a formal request to FHWA for New or Revised Access to the Interstate System. The Interstate Access Request, previously known as an Interchange Justification Report, requires the development of an associated Interstate Access Document (IAD). The IAD is a stand-alone document, which must accompany the request from INDOT to FHWA for approval of new or revised access. The IAD must document and demonstrate that reasonable care has been taken in addressing the criteria described in the FHWA Policy on Access to the Interstate System (FHWA Policy) as described in the *State of Indiana Interstate Access Request Procedures* and Section 48-1.03(06). The IAD must confirm that future traffic operations along the affected Interstate corridor will not be adversely affected by the proposed action. Revisions to the FHWA Interstate Access Policy occur periodically to ensure the focus remains on safety, operational, and engineering issues. The entire Interstate System in the state is under jurisdiction of INDOT. Only INDOT, and not a local public agency or private concern, may develop an Interstate Access Request and submit it to FHWA for approval. New or revised access to the Interstate System must be in accordance with the *State of Indiana Interstate Access Request Procedures*. The procedures are available from INDOT’s Designers webpage at [http://www.in.gov/indot/2731.htm](http://www.in.gov/indot/2731.htm).

The requirement for an Interstate Access Request and such FHWA approval applies only for non-tolled Interstate routes and Interstate toll roads where federal-aid funds have been expended or where the tolled sections have been added to the Interstate System under the requirements of 23 USC 139(a). Access to a non-Interstate System freeway or to a new Interstate System highway does not require an Interstate Access Request. INDOT has the authority to approve new or revised access to all other types of routes where federal-aid funds were used to acquire the access control. For this situation, INDOT must obtain the value of the access from the appropriate property owner(s) and either credit the federal share under existing disposal requirements, or determine that the net proceeds can be handled in accordance with 23 USC 156. INDOT may request FHWA advice or assistance on the acceptability of these types of new or revised access if desired.
48-1.03(02) Actions Requiring an Interstate Access Request

Actions that require INDOT to develop and FHWA to approve an Interstate Access Request include:

1. Establishing a new Interstate-to-Interstate or Interstate-to-freeway (system) interchange;

2. Major modification of a system interchange; e.g., adding new ramp(s), removing ramp(s) from service, significantly relocating tie-in points (terminals) on the freeway, or, where all movements are not currently accommodated, adding ramps to provide for all movements;

3. Upgrading an Interstate-to-non-freeway (service) interchange to an Interstate-to-freeway or Interstate-to-Interstate system interchange;

4. Establishing a new or revised partial interchange on the Interstate of any form;

5. Establishing a new Interstate-to-non-freeway (service) interchange;

6. Modifying an existing Interstate-to-non-freeway (service) interchange, e.g., adding a new ramp, removing a ramp from service, significantly relocating tie-in points (terminals) on mainline freeway or crossroad, or adding or significantly altering collector-distributor (C-D) road elements;

7. Removing select access points or ramps or an entire interchange from service;

8. Changing the essential type of interchange, e.g., replacing conventional diamond with partial cloverleaf;

9. Changing the essential form of a ramp, e.g., directional, semi-directional, loop, or diagonal;

10. Changing intersection control at ramp terminals where the change may affect mainline Interstate flow, even if a new access point to the Interstate is not created. For example, the conversion of a conventional diamond interchange to a diverging diamond interchange or single point diamond interchange is a change that may affect mainline Interstate flow.
11. New or revised locked-gate access or access via locked gates for privately or publicly employed personnel. Locked-gate access is limited to use by utility or INDOT personnel and not the general public; or

12. Establishing new or revised access not explicitly listed above, e.g., those rising to a level beyond incidental work. These instances should be coordinated with the Traffic Engineering Division Corridor Development Office.

48-1.03(03) Actions Not Requiring an Interstate Access Request

The following action do not require INDOT to develop an Interstate Access Request; however, traffic analysis to support the action is essential and should be included in the project file.

1. Changing a single-lane freeway exit or entrance to a two-lane freeway exit or entrance;

2. Widening a single-lane on- or off-ramp (ramp proper) to two or more lanes;

3. Widening (adding auxiliary lanes to) an on- or off-ramp at its intersection with a crossroad (at-grade terminal) to provide two or more intersection approach lanes;

4. Implementing traffic signal control at the ramp terminals;

5. Realigning ramp (minor changes to horizontal or vertical alignment);

6. Converting a tapered on-ramp design (single-lane or multi-lane) or a tapered single-lane off-ramp design to a parallel design.

7. Converting a parallel design off-ramp (single-lane or multi-lane) to a tapered design multi-lane off-ramp with option lane;

8. Increasing the length of an on-ramp acceleration lane or an off-ramp deceleration lane;

9. Adding one or more continuous auxiliary lanes between two adjacent interchange ramps. An operational analysis is required for this action. The NEPA process must be complied with for potential significant environmental impacts from the added lanes (noise, air quality, additional right-of-way, etc.); or

10. Other minor actions not explicitly listed above.
Regardless of the need for an Interstate Access Request, a traffic operational analysis should be conducted. INDOT will informally consult with the appropriate FHWA Transportation Engineer even if such project is not subject to FHWA oversight.

48-1.03(04) Programmatic Agreement for Interstate Access Requests

INDOT and FHWA entered into a Programmatic Agreement for Interstate Access Requests in October 2016. This agreement allows INDOT to conduct the necessary review and assessment of the justification and documentation substantiating certain proposed changes in Interstate System access. The agreement also allows INDOT to make a determination of engineering and operational acceptability (EOA) for proposed changes and request expedited FHWA approval.

INDOT’s determination of EOA is limited to:

1. New and major modifications to existing freeway-to-crossroad (service) interchanges and
2. Completion of basic movements at existing partial interchanges.

The Programmatic Agreement does not include:

1. New or modified freeway-to-freeway (system) interchanges;
2. New partial interchanges;
3. Closure of individual access points that result in partial interchanges or closure of entire interchanges; and
4. Locked gate access.


48-1.03(05) Coordination with National Environmental Policy Act (NEPA) Requirements

When a federal agency is required to make an approval action, regardless of the funding source, the NEPA process must be followed. Since FHWA approves INDOT’s Interstate Access Requests, the NEPA process must be followed when developing new or revised Interstate access. The NEPA process should proceed concurrently with development and analysis of (existing) Interstate access alternatives. The intention is to eliminate early alternatives that would not be acceptable from a transportation and safety operations standpoint. The final decision on a preferred and selected alternative is made as part of the NEPA process. FHWA final Interstate Access Request approval can only be obtained after completion of the NEPA process.
48-1.03(06) General Steps in Revising or Adding Access to the Interstate System

There are five sequential steps in the process for INDOT to secure authorization from FHWA to change Interstate System access. These proposed actions usually require an Environmental Impact Statement (EIS) or an Environmental Assessment (EA) to complete the NEPA process. The first two steps effectively take place as a forerunner to the formal Interstate Access Request process. The steps are outlined in the State of Indiana Interstate Access Request Procedures document and are summarized below.

1. **Framework for Project Scope.** Establish the framework for scope of study relative to alternatives’ analysis, and record that in a concise Framework Document. At the start of access request process, the INDOT project team will meet with FHWA to identify any special process and operational requirements. The Traffic Engineering Division Corridor Development Office oversees development of all Interstate Access Request activities. The FHWA Project Delivery Team Leader will serve as INDOT’s point of contact for this process of developing and screening alternatives. The FHWA Project Delivery Team’s assigned Transportation Engineer will represent FHWA in providing opinion and review of alternatives from an engineering and transportation operations standpoint.

2. **Alternatives Analysis and Selection.** Carry out alternatives’ analysis, and document those activities and findings in a report - the Alternative Evaluation Report. Its findings will indicate if the Interstate Access Request and associated Interstate Access Document (IAD) are required. If an Interstate Access Request is not required for an interchange modification project, an Alternative Evaluation Report will still be required to identify the site, background information, deficiencies, alternatives and proposals. The report will evaluate traffic operations and safety performance of each alternative regarding the interchange itself and the mainline interstate.

3. **Interstate Access Request Determination.** Determine whether an Interstate Access Request to FHWA and its associated IAD are required, and if so, prescribe the nature or scale of that IAD.

4. **Draft Interstate Access Document Submittal.** Produce the IAD, and transmit to FHWA from INDOT the request for engineering and operational acceptability along with that supporting IAD. The draft document will focus on the points of the FHWA Policy. FHWA’s Concept Approval is given with the understanding that the proposal will be that which is reflected in the final NEPA document, either CE, Finding of No Significant Impact (FONSI), or Record of Decision (ROD). This is the first of two approval phases.
5. Request for Final Interstate Access Approval. Transmit to FHWA from INDOT the request for full and final approval, following NEPA approval (CE, FONSI, or ROD). FHWA will respond in writing within four weeks indicating approval or denial of INDOT’s formal request for new or revised access. This is the second of the dual approval phases.

48-1.03(07) Content of the Interstate Access Document

The Interstate Access Document (IAD) that accompanies the Interstate Access Request must address the policy requirements (criteria) outlined in the FHWA Policy. The Programmatic Agreement includes the criteria while the State of Indiana Interstate Access Request Procedures document outlines the analysis and documentation requirements for requesting changes to Interstate System access. The IAD serves as the record of that analysis in the form of answers to the FHWA Policy criteria.

The criteria will be the focus of attention in the IAD and must be directly addressed. Other background information may be presented to supplement that core element. A clear description of the proposed new or revised access should be presented, generally in narrative form directing the reader to sketch-plan drawings. All relevant notes, summary printouts, and/or electronic input/output files of traffic operation and safety analysis should be appended to the IAD document, be they from HCM / HCS, microsimulation, HSM / IHSDM or other method of analysis.

Background information should be included that may help explain or support the proposal, including a description of the influence of the area’s regional transportation network, and any known areas of concern, e.g., environmental, safety, related projects, and long-range transportation plans. A crash analysis summary must be included. The analysis must include a summary of crash data for the previous three-year period. There must be a discussion of the anticipated safety impact the access change will have on the Interstate-route mainline and interchange ramps. The analysis must demonstrate that the access change will not compromise safety. The recommended alternative should include plans with, at a minimum, a table of basic geometric design criteria, horizontal and vertical alignment, curve data, typical sections, signing, and pavement markings. Any necessary design exceptions should desirably be identified. In addition, the total estimated cost of the project should be provided. A complex urban project may require a conceptual-stage signing plan if determined to be necessary by FHWA and INDOT.

48-1.03(08) FHWA Approval

Approval is required from the FHWA Washington, D.C., Headquarters office (HQ) for the major types of new or revised access requests listed below. The Final IAD must be sent by INDOT to
the FHWA Indiana Division office for those actions of a significant nature requiring coordination with HQ. Advance coordination with HQ will be necessary for certain complex or controversial projects. INDOT should coordinate directly with the Division office, specifically, the appropriate Transportation Engineer for all projects.

1. **FHWA Approval by HQ.** HQ approval is required for the types of Interstate System new or revised access as follows:
   a. establishing a new freeway-to-freeway (system) interchange;
   b. major modification of a freeway-to-freeway interchange; or
   c. establishing a new partial interchange of any form.

2. **FHWA Approval by Division Office.** The Final IAD must be sent to the Division office for approval for the types of Interstate system new or revised access as follows:
   a. establishing a new freeway-to-non-freeway (service) interchange;
   b. modifying an existing freeway-to-non-freeway interchange configuration;
   c. establishing locked-gate access; or
   d. removing ramps or interchanges from service.

3. **Time Limits of FHWA Approval.** An affirmative determination by FHWA of safety, operational, and engineering acceptability for proposals for new or revised access points to the Interstate System should be reevaluated whenever a significant change in conditions occurs (e.g., land use, traffic volumes, roadway configuration or design, or environmental commitments). Proposals may be reevaluated if the project has not progressed to construction within 3 years of receiving an affirmative determination of engineering and operational acceptability (23 CFR 625.2(a); see also 23 CFR 771.129). If the project is not constructed within this time period, FHWA may evaluate whether an updated IAD based on current and projected future conditions is needed to receive either an affirmative determination of safety, operational, and engineering acceptability, or final approval if all other requirements have been satisfied (23 U.S.C. 111, 23 CFR 625.2(a), and 23 CFR 771.129). The NEPA document re-evaluation period is also 3 years (23 CFR 7714.129).

48-1.04 Grade Separation Versus Interchange

Once it has been determined to provide a grade-separated crossing, the need for access between the two roadways with an interchange must be determined. The following lists several guidelines to consider when determining the need for an interchange:
1. **Functional Classification.** Interchanges should be provided at all freeway-to-freeway crossings. On fully access-controlled facilities, interchanges should be provided with all major highways, unless this is determined inappropriate for other reasons. Interchanges to other highways should be provided if practical.

2. **Site Conditions.** Site conditions which may be adaptable to a grade separation may not always be conducive to an interchange. Restricted right-of-way, environmental concerns, rugged topography, etc., may restrict the practical use of an interchange.

3. **Interchange Spacing.** When interchanges are spaced farther apart, freeway operations are improved. Spacing of urban interchanges between interchange crossroads should not be less than 1 mile. This should allow for adequate distance for an entering driver to adjust to the freeway environment, to allow for proper weaving maneuvers between entrance and exit ramps, and to allow for adequate advance and turnoff signing. In urban areas, a spacing of less than 1 mile may be developed by grade-separated ramps or by collector-distributor roads. In rural areas, interchanges should not be spaced less than 3 miles apart on the Interstate system or 2 miles on other systems.

4. **Access.** Interchanges may be required in areas where access availability from other sources is limited, and the freeway is the only facility that can practically serve the area.

5. **Operations.** Grade separated facilities without ramps will require all drivers desiring to turn onto the cross road to use other locations to make their desired moves. This will often improve the operations of the major facility by concentrating the turning movements at a few strategically placed locations. However, undue concentration of the turning movements at one location may overload the capacity of the exit or entrance facility.

6. **Overpass vs. Underpass Roadways.** A detailed study should be made at each proposed highway grade separation to determine whether the main road should be carried over or under the crossroad. Often the decision is based on features such as topography or functional classification.
48-2.0 INTERCHANGE TYPE SELECTION

The Traffic Engineering Corridor Development Office determines the interchange type for the site. Typically, the Corridor Development Office will evaluate several types for potential application.

48-2.01 General Evaluation

There are three overall factors that influence the selection of an interchange type:

1. Basic Type. A freeway interchange will be one of two basic types. A “systems” interchange will connect a freeway to a freeway; a “service” interchange will connect a freeway to a lesser facility.

2. Urban/Rural. In rural areas where interchanges occur relatively infrequently, the design can normally be selected strictly on the basis of service demand and analyzed as a separate unit. In urban areas where restricted right-of-way and close spacing of interchanges are common, the type selection and design of the interchange may be severely limited. The operational characteristics of the intersecting road and nearby interchanges will be major influences on the design of an urban interchange.

3. Movements. All interchanges should provide for all movements, even when the anticipated turning volume is low. An omitted maneuver may be a point of confusion to those drivers searching for the exit or entrance. In addition, unanticipated future developments may increase the demand for that maneuver.

Each interchange type should be evaluated considering:

- compatibility with the surrounding highway system;
- route continuity;
- level of service for each interchange element (e.g., freeway/ramp junction, ramp proper);
- operational characteristics (single versus double exits, weaving, signing);
- road user impacts (travel distance and time, safety, convenience and comfort);
- driver expectancy (e.g., exits and entrances to the right);
- geometric design;
- construction and maintenance costs;
- potential for stage construction;
- right-of-way impacts and availability;
- environmental impacts; and
potential growth of surrounding area.

See Figure 48-2A for general guidance for interchange types that are adaptable to freeways based on the functional classification of the intersecting facilities in rural, suburban or urban environments. At intersections other than a freeway-to-freeway, the choice of interchange will likely be limited to a cloverleaf, a diamond, or a variation thereof.

48-2.02 Interchange Types

In Indiana, there are six general interchange types: diamond, full cloverleaf, partial cloverleaf, three-leg, directional, and semi-directional. The following sections discuss these interchange types and the design elements for laying out the interchange, and additional information on alternate interchange designs to fit the site. The final design may be a minor or major modification of one of the general types or may be a combination of two or more general types.

48-2.02(01) Diamond Type Interchanges

The most prevalent type of interchange the diamond. The diamond is a typical service interchange, which links a freeway with a lesser facility (arterial or collector surface street). Intersection control at ramp terminals is typically by way of signalization or signage. Variations of the diamond interchange include:

- Conventional Diamond
- Compressed Diamond (Tight Diamond)
- Diverging Diamond (DDI)
- Single Point Diamond Interchange (SPDI)
- Roundabout Diamond
- Three-Level Diamond

Additional guidance on Diamond Interchange type selection is available from JTRP SPR-3866 Guidelines for selecting Alternative Diamond Interchanges and Performance of Alternative Diamond Interchange Forms” by Purdue University; and HCM 6th Edition, Chapters 23 and 34 procedures for interchange type selection.
Conventional Diamond

The conventional diamond is the simplest and most common interchange type. One-way diagonal ramps are provided in each quadrant and two at-grade intersections at the crossroad. With proper treatments at the crossroad, the diamond interchange can accommodate a wide variety of circumstances in suburban and urban areas where the crossroad operating speeds are 45 mph or less. The diamond is usually the best interchange choice where the intersecting road is not access controlled. See Figure 48-2B for typical conventional diamond interchange schematic.

Advantages of a conventional diamond interchange include:

- All exits from the mainline occur before reaching the crossroad structure and entrances occur after the structure. This conforms to driver expectancy and therefore minimizes confusion.
- All traffic can enter and exit the mainline at relatively high speeds.
- At the crossroad, adequate sight distance can usually be provided, and the operational maneuvers are consistent with other intersections on the crossroad.
- They require less right-of-way than other interchange forms.
- The diamond configuration easily allows modifications to provide greater ramp capacity.
- Their common usage has resulted in a high level of driver familiarity. Typically, it is the least expensive of all interchange types.

Disadvantages of a conventional diamond interchange include:

- Traffic is subject to stop-and-go operations rather than free flow.
- In suburban and urban areas, signalization is generally required at the crossroad intersections. These signals should be interconnected for progression.
- They require right-of-way in all four quadrants of the interchange.
- A diamond has a greater potential for wrong-way entry onto the ramps than, for example, a full cloverleaf. Raised-curb channelization should be provided on the crossroad to minimize the likelihood of driver confusion and wrong-way maneuvers.
Compressed Diamond

A compressed diamond, also known as a tight diamond interchange, is similar to the conventional diamond except that the ramp termini on the crossroad are located near the structure. The compressed diamond typically has the ramp terminal spaced at a minimum from 150 feet to 250 feet. Slip ramps or a combination of frontage roads and slip ramps will accommodate a compressed diamond configuration. This design type is generally only used in urban areas where a diamond interchange is appropriate, but right-of-way or other environmental features preclude the use of the conventional diamond. Although operationally a compressed diamond is similar to a single-point diamond discussed in Section 48-2.02(05), they have significant differences.

Advantages of a compressed diamond interchange include:

- Less right-of-way is required than that for a conventional diamond.
- The open pavement area at the intersection is significantly less than that for a single-point diamond.
- The grade separation structure is significantly smaller than that for a single-point diamond, retaining walls and/or embankments are less expensive, and construction costs are lower.
- The ramp/crossroads intersections operate as two typical intersections, similar to a conventional diamond and, therefore, are less confusing to drivers.
- Slip ramps for one-way frontage roads can be easily incorporated into the design.

Disadvantages of a compressed diamond interchange include:

- Left-turn lanes between the ramp termini usually need to be overlapped (i.e., side-by-side opposing left-turn lanes). Consequently, the cross section of the crossroad is generally wider than a conventional diamond.
- Signal timing and interconnection are necessary in order to eliminate left-turn queues from overlapping each other and causing gridlock.
- Due to the close proximity of the two intersections, the compressed diamond typically will need to operate as a six-phase overlap signal system. Consequently, longer clearance times are required.
- Length of access control on the crossroad may be more extensive than that for a conventional diamond.

Diverging Diamond Interchange (DDI)
The diverging diamond interchange (DDI), also known as a double crossover diamond, is an alternative to the conventional diamond interchange or other alternative interchange forms like a single-point interchange or a partial cloverleaf. A DDI incorporates directional crossovers on either side of the interchange. Cross street traffic is shifted to the opposite (left) side of the street between the signalized ramp intersections. The driver can continue to the left hand entrance ramp without conflicting with opposing through traffic and without stopping. A DDI has fewer conflict points than a conventional diamond interchange and is associated with lower speed, less severe crashes when compared to a conventional diamond interchange. See Figures 48-2C and 48-2D for DDI traffic flow and a conflict point comparison, respectively.

Additional information on DDI planning, design, and operational analysis is available from FHWA publication *Alternative Intersection/Interchange: Informational Report (AIIR)*, Report No. FHWA-HRT-09-060 and FHWA publication *Diverging Diamond Interchange Informational Guide*, Report No. FHWA-SA-14-067.

Advantages of a DDI interchange include:

- Improved operations of turning movements, reduced conflict points, and less severe crashes.
- Free flowing left turns on to freeway entrance ramp.
- Increased interchange capacity. A DDI can alleviate the operational problems of having two closely spaced at-grade intersections on the minor road. In particular, it overcomes the left-turning lane storage problem for drivers wishing to enter the freeway when compare to a conventional diamond interchange.
- Reduced lost time and increased capacity due to two-phase traffic signals.
- Right turns on to the entrance ramp are either free-flow or yield controlled.
- Reduced costs associated with the design and construction time compared to other typical interchange designs. The footprint of the DDI can often fit within the existing right-of-way and on an existing bridge.

Disadvantages of a DDI interchange include:

- Potential driver unfamiliarity with crossover design and merges from left. Additional channelization, signing and pavement markings beyond the levels of a conventional diamond interchange are needed. Additional design consideration needed to minimize driver confusion and the likelihood of wrong-way maneuvers.
- Pedestrian walkways may traverse through the interchange in an unconventional manner. Preferably, the design should provide for pedestrians to cross the minor roadway at adjacent intersections, instead of the ramp terminal intersections if no central pedestrian island between the crossover intersections is present.
May require access control beyond interchange. Driveways and street approaches may have to be relocated to accommodate crossover and reverse curves.

- Does not allow exit ramp to entrance ramp movement.
- May require additional lighting due to unique geometry.
- Is difficult to retrofit for additional capacity.

**Single-Point Diamond Interchange (SPDI)**

The single-point diamond interchange (SPDI), previously referred to as a single-point urban interchange or SPUI, offers improved traffic-carrying capabilities, safer operations, and reduced right-of-way needs under certain conditions when compared with other interchange configurations. The distinguishing feature of this interchange is the convergence of all through and left-turning movements into a single, large signalized intersection area. Figure 48-2E illustrates a schematic of a SPDI.

Advantages of an SPDI include:

- Requires only one intersection instead of two intersections at a typical diamond.
- Allows for better traffic signal progression on the crossroad.
- Potentially increases interchange capacity and alleviates storage problems from two closely spaced intersections on the crossroad.
- Opposing left turns operate to the left of each other so that their paths do not cross each other.
- Requires less right-of-way than any other interchange type.
- Allows left turns to be completed at higher speeds due to typically flatter curves for turning radii at the intersection of the ramps with the crossroad.

Disadvantages of an SPDI include:

- Requires special pavement markings and large channelization islands to guide the left-turning drivers through the intersection.
- Potentially creates traffic delays where pedestrians must cross a significantly wider pavement area (compared to a conventional diamond interchange).
- Requires longer signal clearance times because of wide pavement areas.
- Has a higher cost than the conventional or compressed diamond because of the need for a long, single-span or butterfly structures and the need for retaining walls or reinforced earth walls along the mainline.
**Roundabout Diamond**

Freeway ramp junctions with arterial roads are potential candidates for roundabout intersection treatment. This is especially so if the subject service interchange typically has a high proportion of left-turn flows from the off-ramps and to the on-ramps during certain peak periods, combined with limited queue storage space on the bridge crossing, off-ramps, or arterial approaches. In such circumstances, roundabouts operating within their capacity are particularly amenable to solving these problems when compared with other forms of intersection control.

There are two basic types of roundabout interchanges. The first is a large diameter roundabout centered over or under a freeway. The ramps connect directly into the roundabout, as do the legs from the crossroad. See Figure 48-2F.

The second basic type uses a roundabout at each side of the freeway and is a specific application of closely spaced roundabouts discussed in the previous section. A bridge is used for the crossroad over the freeway or for a freeway to cross over the minor road.

The actual roundabouts can have two different shapes or configurations. The first configuration is a conventional (dumb-bell) one with circular central islands. This type of configuration is recommended when it is desirable to allow U-turns at each roundabout or to provide access to legs other than the cross street and ramps. See Figure 48-2G for configurations.

The second configuration uses tear-drop shaped central islands that preclude some turns at the roundabout. This configuration is best used when ramps (and not frontage roads) intersect at the roundabout. A raindrop central island can be considered to be a circular shape blocked at one end. In this configuration, a driver wanting to make a U-turn has to drive around both raindrop-shaped central islands. This configuration has an additional advantage in that it makes wrong-way turns into the off-ramps more difficult. However, raindrop shapes lack the operational consistency, because one entry will not be required to yield to any traffic. Because of this, an undesirable increase in speed may occur.

Advantages of a roundabout diamond interchange include:

- Reduces number of conflict points and less severe crashes;
- Potentially reduces the queue length on the off-ramps (compared to a signalized intersection);
- Improves traffic operation at the ramp terminal (compared to a signalized intersection); and
- Allows for smoother merging behavior on the freeway and a slightly higher performance at the freeway merge area due to more random ramp traffic (compared with platooned ramp traffic from a signalized intersection).
Disadvantages of a roundabout diamond interchange include:

- Potentially larger footprint (compared to other diamond interchanges);
- Operational impacts due to truck volume;
- Operational impacts due to lack of driver familiarity; and
- Pedestrian discomfort due to traffic operation.

Three-Level Diamond

A three-level diamond interchange includes all of the at-grade intersections on levels separated from the two mainlines. See Figure 48-2H for a three-level diamond interchange schematic.

Advantages of a three-level diamond interchange include:

- It can handle high traffic volumes.
- At-grade intersections are removed from both mainlines, thereby significantly increasing the capacity of the intersection.
- It uses less right-of-way than loop ramps.
- One-way frontage roads can be easily incorporated into the interchange configuration.

Disadvantages of a three-level diamond interchange include:

- To make a left turn, a driver needs to pass through three at-grade intersections and/or traffic signals.
- The additional structures and right of way result in higher construction costs.

48-2.02(02) Full Cloverleaf

Cloverleaf interchanges are used at 4-leg intersections and employ loop ramps to accommodate left-turn movements. Loops may be provided in any number of quadrants. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleaf.

Where two access-controlled highways intersect, a full cloverleaf is the minimum type of interchange design that will suffice. However, a cloverleaf introduces several undesirable operational features such as the double exits and entrances from the mainline, the weaving between entering and exiting vehicles with the mainline traffic and, when compared to directional interchanges, the additional travel time and distance for left-turning vehicles. Therefore, a collector-distributor (C-D) road should be considered with a full cloverleaf, or a fully directional interchange should be provided. See Figure 48-21 for typical examples of full cloverleaf interchanges with and without C-D roads. See Section 48-6.03 for a discussion on C-D roads.
Operational experience with full-cloverleaf interchanges has yielded several conclusions on their design. Subject to a detailed analysis on a site-by-site basis, the following generally characterize the design of a cloverleaf:

1. **Design Speed Impacts.** For an increase in design speed, there will be an increase in:
   - travel distance,
   - required right-of-way, and
   - travel time

2. **Loop Radii.** Considering all factors, loops can be practically designed for approximate radii of 180 to 250-ft. The smaller radii are normally used in urban areas while the larger radii are typically used in rural areas.

3. **Loop Geometry.** Circular curve loop ramps are the most desirable geometrically because speeds and travel paths tend to be more constant and uniform. See Section 48-5.03 for ramp design horizontal alignment.

4. **Loop Capacity.** Expected design capacities for single-lane loops range from 800 to 1200 vph and, for 2-lane loops, 1000 to 2000 vph. The higher figures are generally only achievable where the design speed is 30 mph or higher and few trucks use the loop.

   A loop ramp rarely operates with more than a single line of vehicles, regardless of the roadway width, and therefore, the cloverleaf capacity is limited by the loops. Loops may be made to operate with two lanes abreast, but only by careful attention to design of terminals and the design for weaving, which would need widening by at least two additional lanes at the separation structure. To accomplish this type of design, the terminals should be separated by such great distances and the loop radii be made so large that cloverleafs with two-lane loops generally are not economical from the standpoint of right-of-way, construction, costs, and the amount of out-of-direction travel. Loops that operate with two lanes of traffic are more likely to be considered the exception as to the norm. Two-lane loop ramps should not be used where back to back loops are proposed.

5. **Weaving Volumes.** An auxiliary lane is typically provided between successive entrance/exit loops within the interior of a cloverleaf interchange. This produces a weaving section between the mainline and entering/exiting traffic. When the total volume on the two successive ramps reaches approximately 1000 vph, interference increases rapidly with a resulting reduction of the through traffic speed. At these weaving volume levels, a collector-distributor road should be considered.
6. **Weaving Lengths.** The minimum weaving length between the exit and entrance gores of loops on new cloverleaf interchanges without C-D roads or those undergoing major reconstruction should be at least 1000-ft or the distance determined by a weaving analysis, whichever is greater.

Advantages of a full cloverleaf interchange include:
- Eliminates all vehicular stops through the use of merges.
- Eliminates all at-grade intersections and, therefore, eliminates left turns.
- Where right-of-way is reasonably inexpensive and adverse impacts are minimal, a full cloverleaf may be a practical option.

Disadvantages of a full cloverleaf interchange include:
- Requires more right-of-way and is more expensive than a diamond interchange.
- Loops operate at lower speeds. The loops in a cloverleaf result in a greater travel distance for left-turning vehicles (compared to diamond interchanges);
- Violates driver expectancy as half the exits and entrances are located beyond the crossroad structure;
- May introduce signing problems;
- Results in weaving sections. If the sum of traffic counts on two adjoining loops approaches 1,000 vehicles per hour, interference mounts rapidly, which results in a reduction of speed of through traffic. Consideration should be given to adding a collector-distributor road. The use of auxiliary lanes for acceleration or deceleration lanes is an alternative to collector-distributor roads.
- Generally, ramps at diamond interchanges can be easily widened to increase capacity; while, two-lane loop ramps which are rare, would require at least two additional lanes (one on each side) through the separation structure, longer weaving distances and may require a larger loop radius to operate.
- Pedestrian movements along cross streets can be difficult to safely accommodate at cloverleaf interchanges.
- A loop rarely operates with more than a single line of vehicles, and thus has a design capacity of 800 to 1,200 vehicles per hour.
- Potential for increased crash rates due to weaving issues.

**48-2.02(03) Partial Cloverleaf**

Partial cloverleaf (ParClo) interchanges are service interchanges with loops in one, two, or three quadrants. See Figure 48-2J for ParClo interchange schematics. Various configurations include:
1. **ParClo-A Interchange.** A ParClo-A has two-loop ramps are in opposite quadrants and accommodate the left turning vehicle from the cross street on to the freeway. The term ParClo-A refers to the location of the loop ramps in relation to the driver approaching the interchange. In the case of a ParClo-A, the loop on the driver’s side of the approaching the interchange of the freeway is in advance of the cross street. The spacing of the intersections on the cross street are dependent on the radii of the loop ramps. The ParClo-A is constructed with two successive entrance spaced 1,000 to 1,500-ft apart in each direction of the freeway. Traffic from the two entrance ramps (loop ramp and right-turning ramp) may be merged first and then enter the freeway as a single entrance point where traffic volumes warrant or if volumes are small the two separate entrance design maybe used. Two phase signals would be utilized on the cross street intersections if warranted.

2. **ParClo-B Interchange.** This interchange type also has two loop ramps in opposite but different quadrants than the ParClo-A. For the driver on the freeway approaching the ParClo-B interchange, the loop ramp on the right side of the freeway is beyond the cross street.

3. **ParClo-AB Interchange.** Half the interchange functions as a ParClo-A and the other half functions as a ParClo-B. The design with loops in adjacent interchange quadrants causes weaving on the cross street between the loops. An auxiliary lane may be required between the loops to accommodate the weave.

4. **ParClo-AB (Two Quad) Interchange.** This interchange has all ramps developed on one side of the crossroad. Sometimes referred to as a “folded diamond”. This interchange is applicable where there are close parallel restrictions such as railroads, rivers, etc.

5. **Single-Loop ParClo Interchange.** This interchange is used when the volume of one of the left turning movements is extremely heavy. Adding a loop may not increase the capacity at an interchange under certain traffic volumes and/or conditions.

The above configurations are appropriate where right-of-way restrictions preclude ramps in one or more quadrants. They are also advantageous where a left-turn movement can be provided onto the major road by a loop without the immediate presence of an entrance loop from the minor road.

Interchange ramps in only one quadrant have application for an intersection of roadways with low traffic volumes and minimal truck traffic. Where a grade separation is provided due to topography, and truck volumes don’t justify the separation, a single two-way divided ramp of near minimum design usually will suffice.
Ramps should be arranged so that the entrance and exit movements create the least impediment to traffic flow on the major highway. The ramp arrangement should enable major turning movements to be made by right-turn exits and entrances.

Several of the advantages and disadvantages listed for full a cloverleaf also apply to a partial cloverleaf (e.g., geometric restriction of loops).

Specific advantages of a partial cloverleaf interchange include:

- May offer the opportunity to increase weaving distances, depending upon site conditions.
- Often appropriate where one or more quadrants present adverse right-of-way and/or terrain problems.
- May reduce the number of left-turn movements when compared to a diamond interchange.
- ParClo-A and ParClo-B designs with loops in opposite quadrants eliminate the weaving problem associated with full cloverleaf design.

48-2.02(04) Three-Leg

Three-leg interchanges, also known as “T” or “Y” interchanges, are provided at intersections with three legs. Figure 48-2K illustrates examples of 3-leg interchanges with several methods of providing the turning movements. See the AASHTO GDHS for additional variations of the three-leg interchange. The trumpet type is shown in (A) where three of the turning movements are accommodated with direct or semi-direct ramps and one movement by a loop ramp. In general, the semi-direct ramp should favor the heavier left-turn movement and the loop the lighter volume. Where both left-turning movements are fairly heavy, the design in (B) or (C) may be suitable. A fully directional interchange (B) or (C) is appropriate when all turning volumes are heavy, or the intersection is between two access-controlled highways. These would be the most costly type because of the necessary multiple structures. A three-leg interchange should only be considered when future expansion in the unused quadrant is either impossible or highly unlikely. They are very difficult to expand or modify in the future. See Section 48-4.05 for applicable major divergence and branch connection design.

48-2.02(05) Directional and Semi-Directional

The following definitions apply to directional and semi-directional interchanges:

1. Directional Ramp. A ramp that does not deviate greatly from the intended direction of travel. See Figure 48-2L.
2. **Semi-Directional Ramp.** A ramp that is indirect in alignment, yet more direct than loops. See Figure 48-2M.

3. **Fully Directional Interchange.** An interchange where all left-turn and right-turn movements are provided by directional ramps. See Figure 48-2L.

4. **Semi-Directional Interchange.** An interchange where one or more left-turn movements are provided by semi-directional ramps, even if the minor left-turn movements are accommodated by loops. See Figure 48-2M.

Directional or semi-directional ramps are used for heavy left-turn movements to reduce travel distance, to increase speed and capacity and to eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with cloverleaf interchanges. Left-hand exits and entrances violate driver expectancy and, therefore, should be avoided.

Directional or semi-directional interchanges are most often warranted in urban areas at freeway-to-freeway or freeway-to-arterial intersections. They may require less right-of-way than cloverleaf interchanges. A fully directional interchange provides the highest possible capacity and level of service, but it is extremely costly to build because of the multiple-level structure required. Interchanges involving two freeways will almost always require directional layouts. See Section 48-4.05 for applicable major divergence and branch connection design.

**48-3.0 TRAFFIC OPERATIONAL FACTORS**

**48-3.01 Basic Number of Lanes**

The basic number of lanes is the minimum number of lanes designated and maintained over a significant length of a route based on the overall operational needs of that section. The number of lanes should remain constant over significant distances. For example, a lane should not be dropped at the exit of a diamond interchange and then added at the downstream entrance simply because the traffic volume between the exit and entrance drops significantly. Likewise, a basic lane should not be dropped between closely spaced interchanges simply because the estimated traffic volume in that short section of highway does not warrant the higher number of lanes.

**48-3.02 Lane Balance**

Lane balance refers to certain principles which apply at freeway exits and entrances as follows:

1. **Exits.** At exits the number of approach lanes on the highway should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one. An
exception to this principle would be at cloverleaf loop ramp exits which follow a loop ramp entrance or at exits between closely spaced interchanges (i.e., interchanges where the distance between the end of the taper of the entrance terminal and the beginning of the taper of the exit terminal is less than 1500-ft and a continuous auxiliary lane between the terminals is being used). In these cases, the auxiliary lane may be dropped in a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit. Figure 48-4G illustrates a taper type multi-lane exit ramp design. The configuration provides lane balance and increased weaving capacity when a continuous auxiliary lane is present due to its option lane feature.

2. **Entrances.** At entrances the number of lanes beyond the merging of the two traffic streams should be not less than the sum of the approaching lanes minus one. It may be equal to the number of traffic lanes on the merging roadway.

3. **Traveled Way.** The traveled way width of the highway should not be reduced by more than one traffic lane at a time.

The following violate the principle of lane balance:

- dropping two lanes at a 2-lane exit ramp. One lane should provide the option of remaining on the freeway.
- immediately merging both lanes of a 2-lane entrance ramp into a highway mainline without adding at least one additional lane beyond the entrance ramp.
- immediately adding two or more lanes at the same location on a freeway, even if it is in advance of a multi-lane exit ramp. An auxiliary lane (preferably 1500 ft min.) should be included between each successive added lane or each lane reduction.

Figure 48-3A illustrates how to achieve lane balance at the merging and diverging points of branch connections.

### 48-3.03 Weaving Analysis

A freeway facility is comprised of three types of segments: weaving segment, ramp junctions and basic freeway segments. While these segments have different operating characteristics, conditions within particular segment impact the traffic flow conditions of upstream and downstream segments.

Weaving is generally defined as the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices (except for guide signs). Thus, weaving sections are formed when merge areas are closely followed by diverge areas. The term “closely” implies that there is not sufficient distance between the ramp
merge and diverge areas for them to operate independently. The AASHTO GDHS recommends a minimum ramp spacing of 2000 feet between a system and a service interchange and a minimum of 1600 feet between two service interchanges. See Figure 48-3B. The minimum ramp terminal spacing is independent of design speed. When minimum spacing cannot be met, spacing which accommodates the AASHTO GDHS Decision Sight Distance for Avoidance Maneuver C and E should be considered.

Weaving sections require intense lane-changing maneuvers as drivers must access lanes appropriate to their desired exit points. Traffic in a weaving section is, therefore, subject to lane changing turbulence in excess of that normally present on basic freeway sections.

48-3.03(01) Weaving Analysis Characteristics

There are three geometric characteristics of weaving sections that affect its operating characteristics:

- length,
- width
- configuration

Length is the distance between the merge and diverge areas forming the weaving section. Width refers to the number of lanes within the weaving section. Configuration is defined by the way entry and exit lanes are aligned with each other. All of these characteristics have an impact upon the critical lane changing activity of a weaving section.

Figure 48-3C illustrates two ways in which the length of a weaving section may be reasonably measured. These lengths correspond to the 2010 Highway Capacity Manual (HCM) and are defined as follows:

\[ L_S = \text{Short Length; the distance between the end points of any pavement markings that prohibit or discourage lane-changing.} \]

\[ L_B = \text{Base Length; the distance between points in the respective gore areas where the left edge of the ramp travel lanes and the right edge of the freeway travel lanes meet.} \]

Previous versions of the HCM tied weaving length to the specifics of the loop ramp design in a cloverleaf interchange as most weaving sections were part of such interchanges. Modern weaving sections cover a wide range of designs and situations, and a more general definition of length is, therefore, appropriate.
Type A, B and C weaving configurations as defined in the 2000 HCM have been redefined into two categories as either one-sided or two-sided weaving sections.

- **One-sided Weaving Section.** A one-sided weaving section is a weaving section in which no weaving maneuver requires more than two lane changes.
- **Two-sided Weaving Section.** A two-sided weaving section is a weaving section formed by a single-lane on-ramp followed closely by a single-lane off-ramp where the ramps are on opposite sides of the freeway; or any weaving section in which one weaving movement requires three or more lane change.

Most weaving sections are of the one-sided variety. In general, this means that the ramps that define the entry to and exit from the weaving section are on the same side of the freeway—either both on the right (most common) or both on the left.

See **Figure 48-3C** for illustrations of one-sided and two-sided weaving sections.

- Illustration (A) shows a typical one-sided weaving section formed by a one-lane, right side on-ramp followed closely by a one-lane, right-side off-ramp, connected by a continuous auxiliary lane. Each weaving vehicle must make one lane change, as illustrated, and the lane changing turbulence caused is clearly focused on the right side of the freeway.
- Illustration (B) a typical major weaving section. A major weave is formed when one or more entry/exit legs have multiple lanes. This is considered as a one-sided weaving section in which the on-ramp has two lanes. One weaving movement can be made without a lane change (freeway to ramp), while the other (ramp to freeway) requires one lane change.
- Illustrations (C) shows the most common form of two-sided weave scenarios in which a one-lane on-ramp on one side of the freeway (in this case, on the left) is followed closely by a one-lane off-ramp on the other side of the freeway (in this case, on the right). Even though the ramp-to-ramp weaving movement makes only two lane changes, this is still classified as two-sided weaving.
- Illustration (D) shows the less common case in which one of the ramps has multiple lanes. The ramp-to-ramp weaving movement must execute three lane changes.

**48-3.03(02) Weaving Analysis Level of Service (LOS)**

The LOS of the weave should be at least equal to the mainline segment LOS but not lower than one LOS below that of the mainline segment.

Addition information on weave analysis is available in the HCM.
48-3.04 Route Continuity

All highways with interchanges are designated by route number. Desirably, the through driver should be provided a continuous numbered route on which changing lanes is not necessary to continue on the through route. Route continuity is consistent with driver expectancy, simplifies signing and reduces the decision demands on the driver. Interchange configurations should not necessarily favor the heavier traffic movement, but rather, the through route.

48-3.05 Signing and Marking

Proper interchange operations depend partially on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings will increase the clarity of paths to be followed, safety and operational efficiency. The logistics of signing along a highway segment will also impact the minimum acceptable spacing between adjacent interchanges. The Traffic Engineering Division will determine the use of traffic control devices at interchanges.

48-3.06 Uniformity

To the extent practical, all interchanges along a freeway should be reasonably uniform in geometric layout and appearance. Except for highly specialized situations, all entrance and exit ramps should be to the right.

48-3.07 Distance Between Successive Freeway/Ramp Junctions

Especially in urban areas, successive freeway/ramp junctions frequently may need to be placed relatively close to each other. The distance between junctions should provide for vehicular maneuvering, signing and capacity. The ramp-pair combinations are entrance followed by entrance (EN-EN), exit followed by exit (EX-EX), exit followed by entrance (EX-EN), entrance followed by exit (EN-EX). The final decision on the spacing between freeway/ramp junctions will be based on the level-of-service criteria and on the detailed capacity methodology in the Highway Capacity Manual. Figure 48-3B shows the recommended minimum ramp terminal spacing.

48-3.08 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance to accommodate speed change, increase capacity, and weaving for entering and
exiting vehicles. An auxiliary lane may be dropped at an exit if properly signed and designed. The following statements apply to the use of an auxiliary lane within or near interchanges:

1. **Within Interchange.** Figure 48-3D provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering and through movements.

   The distance between the end of the entrance taper (without the connecting auxiliary lane) and the beginning of the downstream exit taper is relatively short (e.g., 1500-ft or less), and/or

2. **Between Interchanges.** Where interchanges are closely spaced and an auxiliary lane is warranted at an entrance or exit, the designer should consider connecting the lane to the exit of the downstream interchange or entrance of the upstream interchange to form a continuous auxiliary lane.

   An existing auxiliary lane can be retrofit between an entrance and an exit ramp by extending the auxiliary lane beyond the physical nose of the exit ramp gore to accommodate merging traffic. The exit gore should be visible throughout the length of the auxiliary lane. See Figure 48-3D(c).

Design details for exits and entrances are provided in Section 48-4.0, and design details for lane drops are provided in Section 48-6.02.

### 48-3.09 Safety Considerations

Safety is an important consideration in the selection and design of an interchange. After many years of operating experience and safety evaluations, certain practices are considered less desirable at interchanges nationwide. The following summarizes several major safety considerations.

1. **Exit Points.** Many interchanges have been built with exit points which could not clearly be seen by approaching drivers. Decision sight distance should be provided where practical at freeway exits. There should be a clear view of the entire exit terminal, including the exit nose. See Section 48-4.01 for the application of decision sight distance to freeway exits.

   Where traffic warrants for cloverleaf and various configurations of ParClo interchanges, a C-D roadway system may be considered where multiple exit and/or entrance ramps may be combined into a single exit/entrance point. Proper advance signing of exits is essential.
2. **Exit Speed Changes.** Freeway exits should provide sufficient distance for a safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve.

3. **Merges.** Rear-end collisions on entrance merges onto a freeway may result from a driver attempting the complicated maneuver of simultaneously searching for a gap in the mainline traffic stream and watching for vehicles in front. An acceleration distance of sufficient length should be provided to allow a merging vehicle to attain speed and find a sufficient gap to merge into.

4. **Driver Expectancy.** Interchanges can be significant sources of driver confusion; therefore, they should be designed to conform to the principles of driver expectation. Left-hand merges are not desirable. It is difficult for a driver entering from a ramp to safely merge with the high-speed left lane on the mainline. Therefore, left exits and entrances should not be used, because they are not consistent with the concept of driver expectancy when they are mixed with right-hand entrances and exits. In addition, exits should not be placed in line with the freeway tangent section at the point of mainline curvature to the left.

5. **Fixed Objects.** Because of traffic operations at interchanges, a number of fixed objects may be located within interchanges, such as signs at exit gores or bridge piers and rails. These should be removed, where practical, made breakaway or shielded with barriers or crash cushions. Horizontal stopping sight distance should be considered. With the minimum radius for a given design speed, the normal lateral clearance at piers and abutments of underpasses does not usually provide the minimum stopping sight distance. Thus, above-minimum radii should be used for curvature on highways through interchanges. See Chapter 49.

6. **Wrong-Way Entrances.** In almost all cases, wrong-way maneuvers originate at interchanges. Some simply cannot be avoided, but many result from driver confusion due to poor visibility, confusing ramp arrangement, poor channelization or inadequate signing. The interchange design must attempt to minimize wrong-way possibilities.

7. **Weaving.** Areas of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, interchanges should be designed without weaving areas or, as an alternative, with weaving areas removed from the highway mainline (e.g., with collector-distributor roads).

8. **Crossroad.** The crossroad at a rural freeway interchange should be a divided roadway through the interchange area.
48-3.10 Capacity and Level of Service

The capacity of an interchange will depend upon the operation of its individual elements as follows:

1. Basic freeway section where interchanges are not present,
2. Freeway-ramp terminals,
3. Weaving segments,
4. Ramp proper,
5. Ramp/crossroad intersections, and

The *Highway Capacity Manual* (HCM) provide techniques for analyzing the capacity and level of service (LOS) for each element listed above. Highway Capacity Software is required for the analysis for mainline interstate, weaving segments, and ramp junction analysis. Use of other transportation analytical software must be compliant with the applicable sections of the HCM and is subject to approval by the Corridor Development Office.

The interchange should operate at an acceptable LOS. The LOS values presented in Tables 53-1 and 54-2A for freeways will also apply to interchanges. The LOS of each interchange element should be as good as the LOS provided on the basic freeway section. Interchange elements should not operate at more than one LOS below that of the basic freeway section. In addition, the operation of the ramp/crossing road intersection in urban areas should not impair the operation of the mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange. For State projects, the Corridor Development Office is responsible for conducting the preliminary capacity analyses at interchanges.

48-3.11 Testing for Ease of Operation

The designer should review the proposed design from the driver’s perspective. This involves tracing all possible movements that an unfamiliar motorist would drive through the interchange. The designer should review the plans for areas of possible confusion, proper signing and ease of operation and to determine if sufficient weaving distances and sight distances are available. The designer should have available the peak-hour volumes, number of traffic lanes, etc., to determine the type of traffic the driver will encounter.
48-4.0 FREEWAY/RAMP JUNCTIONS

48-4.01 Exit Ramps

48-4.01(01) Types of Exit Ramps

There are two basic types of exit freeway/ramp junctions - the parallel design and the tapered design. For all new and reconstructed service interchange exit ramps, INDOT’s preferred practice is to use the parallel design for single-lane ramps.

The use of a tapered design requires the approval of the Highway Design and Technical Support Director. Ramp design speeds must be in the middle to upper range of mainline design speed. Existing tapered exit ramp designs may be retained if deemed operationally acceptable and there is not an adverse crash history at the ramp junction.

The designer may consider replacing an existing single-lane taper design with a parallel design where:

1. a ramp exit is just beyond a structure and there is insufficient sight distance available to the ramp gore;

2. a taper design cannot be improved to provide the necessary deceleration distance prior to a sharp curve on the ramp;

3. the exit ramp departs from a horizontal curve on the mainline. The parallel design is less confusing to through traffic and will normally result in smoother operation;

4. the need is satisfied for a continuous auxiliary lane. See Section 48-3.08. If the exiting volume warrants a multi-lane exit ramp, a taper type multi-lane exit may be considered to satisfy weaving capacity and to provide lane balance;

5. the capacity of the at-grade ramp terminal is insufficient and ramp traffic may back up onto the freeway.

See Figures 48-4A and 48-4B for detailed design information for INDOT’s typical single lane, parallel and tapered exit freeway/ramp junctions (service interchanges). See Figures 48-4F and 48-4G for detailed design information for INDOT’s typical multi-lane, parallel and tapered exit freeway/ramp junctions (service interchanges).

For system interchanges, see Section 48-4.04, Major Forks/Branch Connections.
48-4.01(02) Taper Rates

For a parallel-lane exit design, the taper rate applies to the beginning taper of the parallel lane. This distance is typically 300-ft as illustrated in Figure 48-4A.

48-4.01(03) Divergence Angle

The divergence angle is the angle of departure from the mainline traveled way on a freeway to the exit ramp which typically develops a taper along the gore at a rate which ranges from 20:1 to 30:1. The AASHTO GDHS allows the divergence angle to range from 2° to 5°. To provide uniformity through-out Indiana’s highway system, all new and reconstructed service interchange single and multi-lane exit ramps should have a divergence angle of 2°17’26” (25:1 taper) as shown in Figure 48-4A. Exceptions to this practice require approval from the Highway Design and Technical Support Division Director.

48-4.01(04) Deceleration

Sufficient deceleration distance is needed to safely and comfortably allow a vehicle to exit the freeway mainline. For a parallel design (Figure 48-4A), the minimum deceleration length is the minimum length including adjustments for grade, or 800 ft, whichever is greater. See Figures 48-4K and 48-4I for minimum deceleration lengths and adjustment values, respectively. Additional length beyond 800 ft should be added to the parallel segment located adjacent to the 300-ft taper. In restrictive area where it is impractical to extend the parallel segment, the deceleration length may include the first curve downstream from the gore nose, provided the curve radius exceeds 3000 ft and the curve length exceeds 300 ft.

Where a tapered design exit ramp is approved for use, the ramp design speed must be in the middle to upper range of the mainline design speed shown in Figure 48-5A. For a tapered design, the minimum deceleration length is 600 ft. This length encompasses downgrades up to 6% and ramp design speeds up to 50 mph.

48-4.01(05) Sight Distance

The sight distance approaching the gore nose should exceed the stopping sight distance for the through traffic, desirably by 25% or more. Where there are unusual conditions, consider providing decision sight distance to the exit terminal. Extra sight distance is particularly important for exit loops immediately beyond a structure. When measuring for adequate sight distance, ensure that the motorist can see the pavement surface at and beyond the gore nose. Locating the exit terminal
and gore nose where the mainline is on an upgrade provides the best design condition. Do not locate exit terminals near mainline crest vertical curves where the ramp pavement may disappear from the driver’s view.

48-4.01(06) Superelevation

Superelevation for horizontal curves near the mainline/ramp junction must be developed to properly transition the driver from the mainline to the curvature at the exit. The principles of superelevation for open-roadway conditions, as discussed in Chapter 43, should be applied to the mainline/ramp junction. If drainage impacts to adjacent property or frequency of slow-moving vehicles are important considerations, low speed urban criteria may be used if the design speed on the ramp proper is 45 mph or less. The following will apply to superelevation development at exit ramps:

1. $e_{\text{max}}$. On the exit ramp portion of the mainline/ramp junction, the typical $e_{\text{max}}$ is 8%.

2. Superelevation Rate. As discussed in Section 43-3.0, Method 5 is used for open-roadway conditions to distribute superelevation and side friction. Therefore, Figure 43-3A(1) will be used to determine the proper superelevation rate for horizontal curves at exit ramps. The designer will use the ramp design speed and the curve radius to read into the tables to determine “$e$”, subject to $R_{\text{min}}$ for the ramp design speed. The superelevation rate and radii used should reflect a decreasing sequence of design speeds for the exit terminal, ramp proper, and at-grade terminal for diamond ramps.

3. Transition Length. The designer should use the superelevation transition lengths for 2-lane roadways as presented in Figure 43-3A(1) to transition the exit ramp cross slope to the superelevation rate at the PC.

4. Distribution. Depending on the number of lanes rotated and design speed, the superelevation transition length should be distributed such that 70 to 90 % of the length is in advance of the PC and the remainder beyond the PC. However, at freeway/ramp junctions, field conditions may make this distribution impractical, and a different distribution may be necessary. However, it should not be less than 50/50.

5. Axis of Rotation. This will typically be about the centerline of the ramp travelway for two-lane ramps and may be about either the centerline or the outside edge of travelway for single-lane ramps.

48-4.01(07) Cross Slope Rollover
The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance lane, where these two are adjacent to each other. At freeway entrances and exits, the maximum algebraic difference between adjacent lanes and gore areas should not exceed 5%.

See Section 48-4.01(09) for nose definitions.

**48-4.01(08) Shoulder Transition**

The right shoulder of the mainline will be transitioned to the narrower shoulder of the ramp. As illustrated in Figures 48-4A and 48-4B, the shoulder width along the mainline will be maintained until 100-ft before the gore nose or ramp PC. The shoulder width will then be transitioned to the ramp right shoulder width (typically 8 ft). In restricted areas, it is acceptable to provide a 6-ft minimum right shoulder along the entire parallel exit ramp area.

**48-4.01(09) Exit Gore Area**

The term *gore* indicates an area downstream from the intersection point of the mainline and exit shoulders. The gore area is normally considered to be both the paved triangular area between the through lane and the exit ramp, plus the graded area which may extend 300 ft downstream beyond the gore nose.

See Figure 48-4D for exit and entrance ramp gore details. The following definitions will apply:

1. **Painted Nose.** The painted nose, also called the theoretical gore, is the point (without width) where the pavement striping on the left side of the ramp converges with the stripe on the right side of the mainline travelway.

2. **Dimension Nose.** The dimensional nose is a point where the shoulder is considered to begin within the gore area. For exit ramps, the dimension nose is 4-ft wide.

3. **Physical Nose.** The physical nose is the point where the ramp and mainline shoulders converge. The physical nose has a dimensional width of 14-ft.

4. **Gore Nose.** The gore nose is the point where the paved shoulder ends and the sodded area begins as the ramp and mainline diverge from one another. The gore nose has a dimensioned width of 6-ft and does not include the shoulders. The total width of the gore nose including the shoulders should be a minimum 20 ft.
The following should be considered when designing the gore.

1. **Obstacles.** If practical, the area beyond the gore nose should desirably be free of all obstacles (except the ramp exit sign) for at least 100 ft beyond the gore nose. Any obstacles within 300 ft of the gore nose are to be made breakaway or shielded by a barrier. See Section 49-3.0.

2. **Side Slopes.** The graded area beyond the gore nose should be as flat as practical. If the elevation between the exit ramp or loop and the mainline increases rapidly, this may not be practical. These areas will likely be non-traversable, and the gore design must shield the motorist from these areas. At some sites, the vertical divergence of the ramp and mainline will warrant protection for both roadways beyond the gore. See Section 49-3.0.

3. **Cross Slopes.** The paved triangular gore area between the through lane and ramp should be traversable. The cross slope is the same as that of the mainline (typically 2%) from the painted nose up to the dimension nose. Beyond this point, the gore area may be depressed to direct drainage to an inlet or discharge point with cross slopes of 2-4%. See Section 48-4.01(07) and Figure 48-4D for criteria on breaks in cross slopes within the gore area.

4. **Traffic Control Devices.** Signing in advance of the exit and at the divergence should be according to the MUTCD and Section 502-1.0. See Section 502-2.0 for the pavement marking details in the triangular area upstream from the gore nose.

5. **Drainage.** Positive drainage within the gore area should be provided by either directing flow to an inlet or by sheet flow to the outside across the ramp. Slotted drains are prone to clogging, may affect traversability, and should not be used in the gore. See Figure 48-4D for cross information relative to the ramp and adjacent pavement.

**48-4.02 Entrance Ramps**

**48-4.02(01) Types of Entrance Ramps**

There are two basic types of entrance freeway/ramp junctions – the parallel design and the tapered design. Figure 48-4C includes detailed design information for these two entrance freeway/ramp junctions. It is INDOT preferred practice to only use the parallel design on new and reconstructed ramps for single and multi-lane entrance ramps. The parallel design offers several advantages when compared to the taper design, including:

1. Where the LOS for the freeway/ramp merge approaches capacity, a parallel design can be easily lengthened to allow the driver more time and distance to merge into the through traffic.
2. Where the acceleration length needs to be lengthened for grades and or trucks, the parallel design provides longer distances more easily than a taper design.

3. Where there is insufficient sight distance available for the driver to merge into the mainline (e.g., where there are sharp curves on the mainline), the parallel entrance ramp allows a driver to use the side-view and rear-view mirrors to more effectively locate gaps in the mainline traffic.

4. Where there is a need for a continuous auxiliary lane, the parallel-lane entrance can be easily incorporated into the design of the continuous auxiliary lane.

**48-4.02(02) Merge Taper Rates**

For parallel design entrance ramps, the taper at the merge point is 600-ft minimum (50:1). For ramps with high truck volumes, use a 70:1 taper.

**48-4.02(03) Acceleration**

Driver comfort, traffic operations and safety will be improved if sufficient distance is available for acceleration. The length for acceleration will primarily depend upon the design speed of the last controlling horizontal curve on the entrance ramp and the design speed of the mainline. When determining the acceleration length, the designer should consider the following:

1. **Passenger Cars.** Figure 48-4H provides the minimum lengths of acceleration for passenger cars. For both parallel and tapered designs, a portion of the ramp proper may be included in the acceleration lane length where the curve approaching the acceleration lane has a radius equal to or greater than 1000 ft. Parallel ramps must also have a minimum length of curve of at least 200 ft. INDOT’s standard ramp configuration for both parallel and tapered entrance ramps uses a minimum radius of 3819.72 ft (3 degrees) for the approaching curve with a curve length of 200-ft. The acceleration distance is measured from the point of the last controlling curve to the beginning of the merge taper (see Figure 48-4C). Where upgrades exceed 3% over the acceleration distance, the acceleration length should be adjusted according to the values presented in Figure 48-4I.

   INDOT’s acceleration lengths provide sufficient distance for acceleration of passenger cars. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merging area, the available acceleration distance should desirably total 1200 ft, exclusive of the taper, to provide additional merging opportunities.
2. **Trucks.** Where the existing or forecasted truck volumes are ≥10% of AADT or 20 trucks per hour, the Traffic Engineering Division of Corridor Development should be contacted to determine if the truck acceleration distances provided in Figure 48-4J are to be considered in the ramp design. Only parallel entrance ramps designs are acceptable at locations where truck traffic dictates the design. Typical areas where trucks might govern the ramp design will include weigh stations, truck stops and transport staging terminals. At other freeway/ramp entrances, the truck acceleration distances should be considered where there is substantial entering truck traffic and where:

   a. the junction operates at a LOS D or worse,

   b. a significant accident history involving trucks which can be attributed to an inadequate acceleration length, and/or

   c. an undesirable level of vehicular delay at the junction attributed to an inadequate acceleration length.

Where upgrades exceed 2%, the truck acceleration distances may be corrected for grades. Figures 44-2B and 44-2C provide performance criteria for trucks on accelerating grades. Before providing any additional acceleration lane length, the designer must consider the impacts of the added length (e.g., additional construction costs, wider structures, right-of-way impacts).

3. **Horizontal Curves:** The specific application of the acceleration criteria to horizontal curves is as follows:

   a. The design speed of the last horizontal curve on the ramp proper will be determined by open-highway conditions. These are discussed in Section 43-2.0.

   b. For relatively short entrance ramps, the acceleration distance may be determined by that distance needed to accelerate from zero (at the beginning of the ramp) to the mainline design speed. The designer should check to determine if this distance governs.

**48-4.02(04) Sight Distance**

Decision sight distance should desirably be provided for drivers on the mainline approaching an entrance terminal. They need sufficient distance to see the merging traffic so they can adjust their speed or change lanes to allow the merging traffic to enter the freeway. Likewise, drivers on the
entrance ramp need to see a sufficient distance upstream from the entrance to locate gaps in the traffic stream for merging. Section 42-2.0 discusses decision sight distance in more detail.

48-4.02(05) Superelevation

The entrance ramp superelevation should be gradually transitioned to meet the normal cross slope of the mainline. The principles of superelevation for open-roadway conditions, as discussed in Section 43-3.01, should be applied to the entrance design. Section 48-4.01(06) provides the superelevation criteria for exit freeway/ramp junctions which are also applicable to entrance freeway/ramp junctions. This includes $e_{\text{max}}$, superelevation rate, transition lengths, distribution of transition lengths between curve and tangent, and the axis of rotation.

48-4.02(06) Cross Slope Rollover

The cross slope rollover is the algebraic difference between the slope of the through lane and the slope of the entrance lane, where these two are adjacent to each other. At freeway entrances and exits, the maximum algebraic difference between adjacent lanes and gore areas should not exceed 5%.

48-4.02(07) Shoulder Transitions

At entrance terminals, the right shoulder must be transitioned from the narrower ramp shoulder to the wider freeway shoulder. See Figure 48-4C for typical shoulder transition details. In restricted areas, it is acceptable to maintain the 6-ft right shoulder width on the ramp throughout the parallel lane until the beginning of the merge taper with the mainline.

48-4.02(08) Entrance Gore Area

Section 48-4.01(09) provides general design considerations for exit gores and the definitions for various nose types which are within the gore area. The following presents the nose dimensions for entrance gores. See Figure 48-4D for entrance and exit ramp gore details.

1. **Painted Nose.** Also called the theoretical gore, this is the point (without width) where the pavement striping on the left side of the ramp converges with the stripe on the right side of the mainline travelway.

2. **Dimension Nose.** The dimension nose width for entrance ramps is 2 ft.
3. **Physical Nose.** The physical nose has a dimensioned width of 14 ft.

4. **Gore Nose.** The gore nose has a dimensioned width of 6 ft and does not include the shoulders. The total width of the gore nose including the shoulders should be at a minimum of 20 ft.

### 48-4.03 Multi-Lane Terminals

Multi-lane terminals may be considered when the capacity of the ramp is too great for single-lane operation. Typically multi-lane ramps are more effective at improving capacity for off-ramps than on-ramps. In both cases, the overall increase in capacity is contingent upon downstream conditions.

The design should consider the following elements for a multi-lane terminal:

1. **Two-Lane Off-Ramp.** A multi-lane off-ramp typically provides more capacity than a single-lane ramp as flow increases through the diverge area. However, the diverge area is controlled by the capacity of the exiting roadway.

2. **Two-Lane On-Ramp.** A multi-lane on-ramp will typically achieve a merge with less turbulence and a higher LOS but will not increase the capacity of the merge, which is controlled by the downstream freeway segment. Although the capacity of a two-lane on ramp is approximately double that of a single lane ramp, it is unlikely that a two-lane ramp can accommodate more than 2250 to 2400 passenger cars per hour through the merge itself. Longer acceleration lanes associated with two-lane ramps results in less turbulence as ramp vehicles enter the freeway traffic stream. This leads to lower densities on the influence area and higher flows in the ramp lanes.

3. **Lane Balance.** Lane balance at the freeway/ramp junction should be maintained. See Section 48-3.02.

4. **Loop Ramps.** Where the capacity analysis indicates that a single-lane loop capacity is insufficient, consideration should be given to providing either a 2-loop ramp or a direct connection ramp. For 2-lane loop ramps, the designer should consider the following:
   a. Two-lane loop ramps should have a minimum radius of 200 ft (180 ft for restrictive conditions). The loop travel-way should not be less than 30 ft (2 15-ft lanes) and for radii less than 200 ft use a travelway width of 32 ft.
   b. Expected design capacities for single-lane loops range from 800 to 1200 vph and for 2-lane loops, 1000 to 2000 vph.
   c. Enough distance needs to be provided to properly design the exit and entrance for the second lane on the loop.
5. **Entrances.** INDOT’s preferred practice for multi-lane entrance ramps is to use a parallel design. See [Figure 48-4E](#) for parallel multi-lane entrance ramp design details.

6. **Exits.** Multi-lane exit ramps may be either a parallel design or a tapered design with an option lane.
   
a. **Parallel Multi-lane Exit Ramp.** For a parallel multi-lane exit ramp, the first auxiliary lane should be added at least 1500 ft in advance of the exit taper. The total length from the beginning of the first taper to the gore nose should range from 3100 ft for turning volumes of 1500 vph or less upward to 4100 ft for turning volumes of up to 3000 vph. See [Figure 48-4F](#) for parallel multi-lane exit ramp design details.

   Where a ramp splits or forks beyond the painted nose of the exit ramp, two parallel deceleration lanes should be provided prior to the gore nose for the 2500-ft length mentioned above. The exit taper to the parallel deceleration lanes should be a minimum of 300 ft long. This parallel design should also be considered where vehicle storage is anticipated in the ramp lanes and deceleration lanes in advance of the crossroad intersection.

   b. **Tapered Multi-lane Exit Ramp with Option Lane.** The tapered design multi-lane exit ramp design has an option lane that allows a driver to remain on the outside lane of the mainline or to exit onto the inside lane of the exit ramp without a lane change. The option lane feature reduces the number of lane changes and automatically provides a lane-balanced exit. The option lane configuration requires providing an additional auxiliary lane of at least 2000 ft. The total length from the beginning of the first taper to the gore nose should be a minimum of 3100 ft and has the capacity to accommodate turning volumes up to 3000 vph. See [Figure 48-4G](#) for tapered multi-lane exit ramp design details.

   The tapered multi-lane exit ramp may offer advantages to the parallel multi-lane exit ramp design as follows:

   - The tapered multi-lane exit ramp design may be more economical due to its reduced footprint.
   - Environmental impacts and right-of-way acquisition are typically lessened due its reduced footprint.
   - The option lane feature reduces lane weaves and increases weaving capacity.
7. **Signing.** The geometric layout of multi-lane exits must be coordinated with the Traffic Division Office of Traffic Design because of the complicated signing which may be required in advance of the exit.

### 48-4.04 Major Fork/Branch Connections

Major fork (divergence) and branch connections (convergence) are used for system interchanges. See Figures 48-4L and 48-4M for typical design details for a major forks and branch connections, respectively.

The designer should consider the following when designing major fork and branch connections:

1. **Lane Balance.** The principle of lane balance should be maintained. See Section 48-3.02.

2. **Divergence Point.** Where the alignments of both roadways are on horizontal curves at a major fork, the painted nose of the gore should be placed in direct alignment with the centerline of one of the interior lanes. This provides a driver in the center lane the option of going in either direction. See Figure 48-4L, schematics A, B and C. Where one of the roadways is on a tangent at a major fork, the gore design should be similar to freeway/ramp multi-lane exit. See Figure 48-4L, schematic D.

3. **Nose Width.** At the painted nose of a major fork, the lane should be at least 24-ft wide but preferably not over 28 ft. The widening from 12 ft to 24 ft should occur within a distance of 1000 ft to 1800 ft. See Figure 48-4L, schematic A.

   If a design hourly volume of greater than 1500 vph is anticipated on the exit ramp at a major fork on a systems interchange, the exit deceleration lanes, exclusive of the exit tapers, should begin approximately 1 mile before the painted gore nose, but not less than 2700 ft.

4. **Branch Connection.** A branch connection forms when two separate multilane freeway ramps or routes converge to form a single freeway route.

   Traffic demand may indicate that the number of lanes beyond the convergence point should equal the combined total number of lanes on the two approach roadways. See Figure 48-4M, schematic A. Otherwise, the number of lanes downstream from the point of convergence may be one less than the combined total on the two approach roadways. When merging, a full lane width should be carried for at least 1000 ft beyond the painted nose and tapered at a minimum of 50:1, preferably at 70:1. See Figure 48-4M, schematic B.
There are no effective models of performance for a major merge area (HCM v6.0, Chapter 14). Therefore, analysis is limited to checking capacities of the approaching legs and the departing freeway. Problems in major merge areas generally result from insufficient capacity of the downstream freeway segment.

Lane drops should be in accordance with Section 48-6.02, Freeway Lane Drops. Because the outer lane from the roadway entering from the left is the low speed lane for that roadway and the inside lane from the roadway entering from the right is the high speed lane for that roadway, turbulence is likely at the convergence point. Consideration should be given to providing more than the 1000-ft minimum to alleviate the turbulence to the extent practical.

48-5.0 RAMP DESIGN

48-5.01 Design Speed

The ramp design speeds may vary. The designer should use the acceptable ranges listed for the ramp types listed below and Figure 48-5A to determine the ramp design speed based on the design speed of the mainline:

1. **Freeway/Ramp Junctions.** The design speeds in Figure 48-5A apply to the ramp proper and not to the freeway/ramp junction. Freeway/ramp junctions are designed using the freeway mainline design speed.

2. **At-Grade Terminals.** If a ramp will be terminated at an at-grade intersection with a stop or signal control, the design speeds in the figure may not be applicable to the ramp portion near the intersection. For additional information on the design speed selection near at-grade intersections, see Chapter 46.

3. **Variable Speeds.** The ramp design speed may vary based on the two design speeds of the intersecting roadways. Higher design speeds should be used on the portion of the ramp near the higher-speed facility while lower speeds may be selected near the lower-speed facility. When using variable design speeds, the maximum speed differential between controlling design elements (e.g., horizontal curves, reverse curves) should not be greater than 10 to 20 mph. The designer needs to ensure that sufficient deceleration distance is available between design elements with varying design speeds (e.g., two horizontal curves).

4. **Ramps for Right Turns.** Design speeds for right-turn ramps are typically in the mid- to high range. This includes, for example, a diagonal ramp of a diamond interchange.
5. **Loop Ramps.** Design speeds in the high range are generally not attainable for loop ramps. Minimum values usually control. For mainline design speeds greater than 50 mph, the loop design speed should not be less than 20 mph. However, design speeds greater than 30 mph will require significantly more right-of-way and may not be practical in urban areas. Normally, a loop should not be designed for a speed greater than 35 mph. Arterial loop ramp radii should desirably be greater than 150 ft.

6. **Semi-direct Connections.** Design speeds between the mid- to high ranges should be used for semi-direct connections. Design speeds greater than 50 mph are generally not practical for short, single-lane ramps. For 2-lane ramps, values in the mid- to high ranges should be used.

7. **Direct Connections.** For direct connections, the design speed should be in the mid to high range.

48-5.02 **Cross Section [Rev. Feb. 2019]**

See Figure 48-5B for single and Figure 48-5C for multilane ramp typical cross sections for tangent and for superelevated scenarios. The following will also apply to the ramp cross sections:

1. **Width.** The minimum paved width of a 1-way, 1-lane ramp will be 28 ft. The 28-ft width includes a 4-ft left shoulder, an 8-ft right shoulder and a 16-ft travelway. Multi-lane ramp widths should be in multiples of 12 ft, with a 4-ft wide left shoulder and a 10-ft wide right shoulder. The guardrail offset from the edge of shoulder should be 2 ft. The bridge railing offset should be 1’-8”. Full-depth paving equal to the ramp pavement thickness should be provided on the shoulders because of frequent use of shoulders for turning movements and passing stalled vehicles.

   The designer must request approval from the Department to reverse the left and right shoulder widths to provide additional sight distance for ramps that have tight or prolonged curves to the left.

2. **Pavement Design.** Loop ramps and other ramps with curve radii less than or equal to 300-ft should be designed with full-depth pavement for the entire 28-ft width. For ramps with curve radii greater than 300-ft, only the 16-ft traveled way will typically have a full-depth pavement structure. Outer connector ramps at a cloverleaf interchange or the ramps at a diamond interchange should have full-depth shoulders. For additional pavement design information, see Chapter 304.

3. **Cross Slope.** On a tangent section of a single lane ramp, the cross slope of the traveled way and the left shoulder match, typically at 2%. The right shoulder cross slope is typically 4%.
For all superelevated ramps, the entire ramp width, including the shoulders, should have the same cross slope. The cross slopes of multi-lane ramps are the same as the cross sectional elements of the freeway mainline typical tangent section. See Figure 48-5B for single lane sections and Figure 48-5C for multi-lane sections.

4. **Curbs.** In general, curbs should not be used on ramps. However, mountable curb may be used for drainage or to prevent erosion on steep embankment slopes. See Section 49-3.04 for additional curbing information. Curbs may be placed at the edge of the roadway of a ramp on a low speed facility if approved by the Department.

5. **Bridges and Underpasses.** The full paved width of the ramp should be carried over a bridge or beneath an underpass. The clear width under an underpass should also include the clear zone.

6. **Side Slopes/Ditches.** Side slopes and ditches should meet the same criteria as for the mainline. See Section 45-3.0 and Section 45-8.0 for additional information on the design of these elements.

7. **Clear Zones.** The clear zone from the edge of the traveled way portion of the ramp will be determined from Figure 49-2A. The design ADT will be the directional ADT on the ramp.

8. **Barriers.** Whenever practical, an additional 2-ft should be added to the shoulder width when a roadside barrier is used. Where a barrier is present on a horizontal curve, the designer should determine the barrier impact on horizontal sight distance. See Section 43-4.04.

9. **Right-of-Way.** The right-of-way adjacent to the ramp should be limited access right-of-way.

### 48-5.03 Horizontal Alignment

#### 48-5.03(01) Theoretical Basis

Establishing horizontal alignment criteria for any highway element requires a determination of the theoretical basis for the various alignment factors. These include the side-friction factor (f), the distribution method between side friction and superelevation, and the distribution of the superelevation transition length between the tangent and horizontal curve. For horizontal alignment on the ramp proper, the theoretical basis will be one of the following:

1. **Open-Roadway Conditions:** Chapter 43 discusses the theoretical basis for horizontal alignment assuming open-road conditions. In summary, this includes the following:
   a. relatively low side-friction factors (i.e., a relatively small level of driver discomfort)
b. the use of AASHTO Method 5 to distribute side friction and superelevation

c. relatively flat longitudinal gradients for superelevation transition lengths

d. for a simple curve, depending on the number of lanes rotated, superelevation runoff length may be distributed from 50% to 90% on the tangent and the remainder on the horizontal curve (formerly the 2/3rd – 1/3rd rule). See Figure 43-3F. A spiral curve transition may be considered where there is a steep downgrade with high volumes and a sharp curve with maximum superelevation. Coordination with the Traffic Division Corridor Development Office is required prior to utilizing a spiral curve transition. See the AASHTO GDHS for additional design guidance on spiral curves.

2. Turning Roadway Conditions: Section 46-3.02 discusses the theoretical basis for horizontal alignment assuming turning roadway conditions. In summary, this includes the following:

   a. higher side-friction factors than open-road conditions to reflect a higher level of driver acceptance of discomfort.
   b. a range of acceptable superelevation rates for combinations of curve radii and design speeds to reflect the need for flexibility to meet field conditions for turning roadway design.
   c. the allowance of some flexibility in superelevation transition lengths and in the distribution between the tangent and curve.

For interchange ramps, the selection of which theoretical basis to use will be based on the portion of the ramp under design, including:

- freeway/ramp junction
- ramp proper (directional ramps)
- ramp proper (loop ramps)
- ramp terminus (intersection control)
- ramp terminus (merge control)

In addition, several general controls will dictate horizontal alignment on interchange ramps. The following sections discuss all horizontal alignment criteria for ramps.

48-5.03(02) General Controls

The following will apply to the horizontal alignment of all ramp elements:
1. **Superelevation Rates (Rural).** For non-loop ramps in rural areas, the superelevation rate will be based on an $e_{\text{max}} = 8\%$ and open-road conditions. See Figure 43-3A3 for specific superelevation rates based on ramp design speed and curve radius.

2. **Superelevation Rates (Urban).** For ramps in urban areas, the superelevation rate will be based on an $e_{\text{max}}$ of 4%, 6% or 8%, depending on site conditions. For open-roadway conditions, $e_{\text{max}} = 8\%$ should be used. Figure 43-3A2 presents specific superelevation rates for $e_{\text{max}} = 6\%$ and Figure 43-3A1 for $e_{\text{max}} = 4\%$ using open-roadway conditions.

3. **Superelevation Transitions.** Open-road conditions, as discussed in Section 43-3.0, will also apply for transitioning to and from the needed superelevation on ramps. This includes the relative longitudinal gradients presented in Figure 43-3E. The methodology presented in Section 43-3.0 is used to calculate the superelevation runoff and tangent runout distances with the following modifications.

   a. **One-Lane Ramps.** When rotated about the centerline, the width of rotation (W) is 12 ft or one-half the travelway width, whichever is greater.

   b. **Two-Lane Ramps.** The width of rotation (W) is assumed to be one-half of the widest travelway, which is determined by the minimum radius ($R = 180\text{-ft}$) for the lowest ramp design speed ($V = 25\text{ mph}$) $(0.5 \times 27 = 13.5\text{-ft})$.

   A 60-ft vertical curve should be provided at superelevation diagram P.I. locations.

4. **Minimum Length of Design Superelevation.** The designer should not superelevate curves on ramps such that the design superelevation rate is maintained on the curve for a very short distance. As a general rule, the minimum distance for design superelevation should be about 120 ft. This distance corresponds to the 60-ft vertical curve provided at the superelevation diagram P.I. locations.

5. **Axis of Rotation.** This will typically be about the centerline of the ramp travelway for two-lane ramps and may be about either the centerline or the outside edge of travelway for single-lane ramps.

6. **Shoulder Superelevation.** The criteria presented in Section 43-3.06(01) High Side Shoulder and Section 43-3.06(02) Low Side Shoulder for superelevating shoulders on conditions will not apply to superelevated curves on ramps. When superelevated, the entire ramp width, including shoulders, should have the same cross slope. See Figures 48-5B, Single Lane Ramp Typical Section and 48-5C Multi-Lane Ramp Typical Section.
7. **Reverse Curves.** To meet restrictive right-of-way requirements, ramps may be designed with reverse curves. Desirably, these reverse curves should be designed with a normal tangent section between. For ramps, however, it is often necessary to provide a continuously rotating plane between the reverse curves. If a continuously rotating plane is used, the distance between the PT and the succeeding PC should be the sum of the respective superelevation runoff lengths located on the tangent. It is not acceptable for the PT and PC to be coincident as this would not allow for an adequate transition of superelevation between the curves. See Section 43-3.0 for more information on superelevation at reverse curves.

8. **Sight Distance.** Section 43-4.0 presents the criteria for sight distance around horizontal curves based on the curve radii and design speed. These criteria also apply to curves on ramps. There should be a clear view of the entire exit terminal, including the exit nose and a section of the ramp roadway beyond the gore.

**48-5.03(03) Freeway/Ramp Junctions**

Horizontal alignment at freeway/ramp junctions is based on open-road conditions. This is discussed in Section 48-4.0.

**48-5.03(04) Ramp Proper (Directional Ramps)**

Directional ramps refer to those ramps which are relatively direct in their alignment. These include ramps at diamond interchanges, the outer ramps at cloverleaf interchanges and ramps at directional and semi-directional interchanges.

The ramp proper, for the purpose of horizontal alignment, is considered to begin at the gore nose on exit ramps and to end approximately 200 ft before the dimension nose on entrance ramps. See the discussion in Section 48-5.03(01) to determine whether open-road conditions or turning roadway conditions apply to the horizontal alignment on directional ramps.

**48-5.03(05) Ramp Proper (Loop Ramps)**

Loop ramps are those ramps on the interior portions of cloverleaf and partial cloverleaf interchanges. The ramp proper is considered to begin at approximately the physical nose on exit ramps and to end at approximately the physical nose on entrance ramps. Because of the normally restrictive conditions for loop ramps, the curve radii are typically less than 300-ft. Therefore, it is desirable to use open-road conditions for horizontal alignment; although, typically, it is more practical to use turning roadway conditions. See the discussion on transition curves in Section 48-2.02(02) and 48-5.03(02).
48-5.03(06)  Ramp Terminus (Intersection Control)

Interchange ramps may end at at-grade intersections. These may be stop control or signal control. If horizontal curves on the ramps are relatively close to the intersection, a design speed for the curve should be selected which is appropriate for expected operations at the curve. For these curves, the radius will determine whether open-road or turning roadway conditions apply. For R ≥ 300-ft, use open-road conditions. For R < 300-ft, open-road conditions are desirable; turning roadway conditions are acceptable.

48-5.03(07)  Ramp Terminus (Merge Control)

Interchange ramps may terminate with a merge into the intersecting road. The horizontal alignment at the ramp merge (or junction) will typically be based on open-road conditions. Profiles of highway ramp terminals should desirably be designed with a platform on the ramp side of the approach nose or merging end. This platform should be at least 200 ft in length. It should have a profile that does not greatly differ from that of the adjacent traffic lane.

48-5.04  Vertical Alignment

48-5.04(01)  Grades

Maximum grades for vertical alignment on ramps cannot be as definitively expressed as those for highway mainline. General values of limiting gradients are 3% to 5% but, for any one ramp, the selected gradient is dependent upon a number of factors. These include the following:

1. The flatter the gradient on the ramp, the longer it will be. At restricted sites (e.g., loops), it may be necessary to provide a steeper grade to shorten the length of ramp.

2. The steepest gradients should be designed for the center portion of the ramp. Freeway/ramp junctions and landing areas at at-grade intersections should be as flat as practical.

3. Short upgrades of as much as 5% do not unduly interfere with truck and bus operations. Consequently, for new construction it is desirable to limit the maximum gradient to 5%.

4. Downgrades on ramps should follow the same guidelines as upgrades. They may, however, safely exceed these values by 1%, with 6% considered to be a maximum. The 6% downgrade
should only be used in extreme conditions and where restrictive geometric elements are clearly visible to the driver.

5. The ramp grade within the freeway/ramp junction up to the physical nose should be approximately the same grade as that provided on the mainline. However, adequate sight distance is more important than grade control. See Figure 48-5A for desirable maximum grades for various design speeds.

48-5.04(02) Vertical Curvature

Vertical curves on ramps should be designed the same as those on the mainline. At a minimum, they should be designed to meet the stopping sight distance criteria. The ramp profile often assumes the shape of the letter S with a sag vertical curve at the lower end and with a crest vertical curve at the upper end. In addition, the vertical curvature of the ramp should be compatible with that of the mainline up to the physical nose. Where a crest or sag vertical curve extends onto the freeway/ramp junction, the length of curve should be determined using a design speed intermediate between those on the ramp and the highway. See Chapter 44 for details on the design of vertical curves.

48-5.05 Roadside Safety

The criteria in Chapter 49 (e.g., clear zones, barrier warrants) will apply to the roadside safety design of interchange ramps. One special situation is the requirement for a median barrier between adjacent on/off ramps (e.g., between the outside directional ramp and inside loop ramp for a cloverleaf interchange). This will be determined on a case-by-case basis. This situation typically occurs at full or partial cloverleaf interchanges.

48-5.06 Ramp Location on a Curve

Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve locations are necessary, only parallel entrance and exit ramps should be used. Ramp entrance tapers should be the same length as if on a tangent section with the width of the taper pro-rated along the length of the taper. The minimum gore nose width of 20-ft should be maintained and the degree of divergence may have to be adjusted depending on the curvature. If the designer has to modify a ramp configuration, documentation of the geometrics to accommodate the required acceleration/deceleration lengths and sight distance should be provided to the satisfaction of INDOT.
48-6.0 OTHER INTERCHANGE DESIGN CONSIDERATIONS

48-6.01 General

The designer should consider the following.

1. **Design Year.** The design year for the minor road intersecting the freeway should be the same as used for the freeway. The termination of other roads and streets in the area may generate a significant increase of traffic on the crossing facility.

2. **Over versus Under.** The decision on whether the freeway should go over or under the cross road is normally dictated by topography. If the topography does not favor one over the other, the following factors can be used as a guide to determine which highway should cross over the other.
   
   a. The designer should consider which alternative will be more cost effective to construct. Some elements are the amount of fill, grading, span lengths, angle of skew, gradients, sight distances, geometrics, constructability, traffic control and costs.
   
   b. One benefit of the cross road going over the freeway is that this may improve the ramp gradients. As drivers exit the freeway, they will normally tend to slow down going up an exit ramp and speed up going down an entrance ramp.
   
   c. The alternative which provides the highest design level for the major road should be selected. Typically, the crossing road has a lower design speed; therefore, the minor road typically can be designed with steeper gradients, lesser widths, reduced vertical clearance requirements, etc.
   
   d. If any crossings and/or structures are planned for a future date, the mainline should go under these future crossings. Overpasses are easier to install and will be less disruptive to the major road when they are constructed in the future.

3. **Underpass Width.** The approach cross section, desirably including clear zones, should be carried through the underpass. Including the clear zone allows for possible expansion in the future with minimal disruption to the overhead structure. In addition, wider underpasses also provide greater sight distance for at-grade ramp terminals near the structure.

4. **Grading.** The designer should consider the grading around an interchange early in design. Properly graded interchanges allow the overpass structures to naturally blend into the terrain. In addition, the designer needs to ensure that the slopes are not too steep to support the bridge
and roadways and that they can support plantings which prevent erosion and enhance the appearance of the area. Flatter slopes also allow easier maintenance.

48-6.02 Freeway Lane Drops

A reduction in the basic number of lanes (lane drops) may be made beyond a principal interchange involving a major fork or at a point downstream from an interchange with another freeway. This reduction may be made provided the exit volume is sufficiently large enough to change the basic number of lanes beyond this point on the freeway route as a whole. Another situation where the basic number of lanes may be reduced is where a series of exits, as in outlying areas of a city, causes the traffic load on the freeway to drop sufficiently to justify the lesser number of lanes. Dropping a basic lane or an auxiliary lane may be accomplished at a two-lane exit ramp or between interchanges. Lane reductions should not be made between and within interchanges simply to accommodate variations in traffic volumes. Instead, auxiliary lanes, as needed, are added or removed from the basic number of lanes.

Figure 48-6A illustrates the recommended design of a lane drop beyond an interchange. The following criteria are important when designing a freeway lane drop.

1. **Location.** The lane drop should occur approximately 2000 ft – 3000 ft beyond the end of entry-ramp taper of the previous interchange. Under restricted conditions, the MUTCD signing distance is acceptable. This distance allows adequate signing and driver adjustments from the interchange, but yet is not so far downstream that drivers become accustomed to the number of lanes and are surprised by the lane drop. In addition, a lane should not be dropped on a horizontal curve or where other signing is required, such as for an upcoming exit.

In urban areas, interchanges may be closely spaced for considerable lengths of highway. In these cases, it may be necessary to drop a freeway lane at an exit. Where this is necessary, it is preferable to drop a freeway lane at a 2-lane exit rather than a single-lane exit. As discussed in Section 48-3.0, a lane should not be dropped at an exit unless there is a large decrease in traffic demand for a significant length of freeway.

2. **Transition.** The desirable transition taper rate is 70:1. The minimum acceptable taper rate being 50:1 (see Figure 48-6A).

3. **Sight Distance.** Decision sight distance (DSD) should be available to any point within the entire lane transition. See Section 42-2.0 for applicable DSD values. When determining the availability of DSD, the desirable height of object will be 0.0 in. (the roadway surface); it is acceptable to use 6-in. This criterion would favor, for example, placing a freeway lane drop within a sag vertical curve rather than just beyond a crest.
4. **Right-Side versus Left-Side Drop.** Right-side freeway lane drops are preferred in urban settings; however, a left-side lane reduction may be more practical in rural settings where truck traffic is usually predominant in the right lanes and overall traffic volumes are less in the left lanes.

5. **Shoulders.** The full-width right shoulder will be maintained through a right-side lane drop. If a left-side lane drop will be used to reduce the number of lanes from three to two, the left shoulder will be reduced from 10 ft (or 12 ft) to 4 ft. The full 10-ft left shoulder should be maintained for a distance of approximately 300 ft beyond the merge point of the dropped lane. The additional distance provides an area to allow a driver, who may have missed the signing, an opportunity to safely merge with the through traffic. The shoulder should then be transitioned from 10 ft to 4 ft over a minimum length of 100 ft. The additional length of shoulder beyond the merge point should be full depth pavement. (See Figure 48-6A).

**48-6.03 Collector-Distributor Roads**

In general, interchanges that are designed with single exits are superior to those with two exits, especially if one of the exits is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve the operational efficiency of the entire interchange.

Collector-distributor (C-D) roads use the single exit approach to improve the interchange operational characteristics. C-D roads:

1. remove weaving maneuvers from the mainline and transfer them to the slower speed C-D roads,

2. provide high-speed single exits and entrances from and onto the mainline,

3. satisfy driver expectancy by placing the exit in advance of the separation structure,

4. simplify signing and the driver decision-making process, and

5. provide uniformity of exit patterns.

C-D roads are most often warranted when traffic volumes are so high that the interchange without them cannot operate at an acceptable LOS, especially in weaving sections. They are particularly advantageous at full cloverleaf interchanges where the weaving between the ramp/mainline traffic can be very difficult. Figure 48-21 illustrates a schematic of a C-D within a full cloverleaf interchange.
C-D roads may be one or two lanes, depending upon the traffic volumes and weaving conditions. Lane balance should be maintained at the exit and entrance points of the C-D road. The design speed of a C-D road usually ranges from 45 to 50 mph; however, it should desirably be within 10 mph of the mainline design speed. The separation between the C-D road and mainline should be as wide as practical but not less than that required to provide the applicable shoulder widths and a longitudinal barrier between the two (e.g., 20 to 25 ft).

**48-6.04 Frontage Roads**

The designer must consider the impact of frontage roads, where present, on interchange design. At some urban interchanges, it may be impractical to separate the intersections of the ramp and frontage road with the crossing road. In these cases, the only alternative is to merge the ramp and frontage road before the intersection with the crossing road. This can apply to either the exit or entrance ramp.

Figure 48-6B provides the basic schematic for this design. This design may only be used in restricted urban areas. The critical design element is the distance “A” between the ramp/frontage road merge and the crossing road. This distance must be sufficient to allow traffic weaving, vehicular deceleration and stopping, and vehicular storage to avoid interference with the merge point. Figure 48-6B also presents general guidelines which may be used to estimate this distance during the preliminary design phase. A number of assumptions have been made including weaving volume, operating speeds and intersection queue distance. Therefore, a detailed analysis will be necessary to firmly establish the needed distance to properly accommodate vehicular operation. See Transportation Research Record 682 *Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design* for additional information.

Distance “B” in Figure 48-6B is determined on a case-by-case basis. It should be determined based on the number of frontage road lanes and the intersection design. This distance is typically determined by the weaving distance from the intersection to ramp entrance. For capacity analysis of the weaving section, see the *Highway Capacity Manual*. Under some circumstances, this distance may be 0.0 ft.

The following summarizes the available options for coordinating the design of the interchange ramps, frontage road and crossing road:

1. **Slip Ramps.** Slip ramps may be used to connect the freeway with 1-way frontage roads before (or after) the intersection with the crossing road. Newly constructed slip ramps to a 2-way frontage road are unacceptable because they may induce wrong-way entry onto the freeway and may cause accidents at the intersection of the ramp and frontage road.
2. **Separate Intersections.** Separate ramp/crossing road and frontage road/crossing road
intersections may be accomplished by curving the frontage road away from the ramp and
intersecting the frontage road with the crossing road outside the ramp limits of full access
control. Figure 48-6C, Typical Access Control for a Partial Cloverleaf Interchange, provides
an illustration of this separation. This treatment allows the two intersections to operate
independently, and it eliminates the operational and signing problems of providing the same
point of exit and entrance for the frontage road and freeway ramp.

Section 45-7.0 discusses overall design criteria for frontage roads (e.g., functional class, outer
separation).

### 48-6.05 Ramp/Crossing Road Intersection

At service interchanges, the ramp will typically end with an at-grade intersection at the cross road. In
general, the intersection should be treated as described in Chapter 46. This will involve a
consideration of capacity and the physical geometric design elements such as sight distance, angle of
intersection, acceleration lanes, channelization and turning lanes. However, several points require
special attention in the design of the ramp/crossing road intersection:

1. **Capacity.** In urban areas where traffic volumes are often high, inadequate capacity of the
ramp/crossing road intersection can adversely affect the operation of the ramp/freeway
junction. In a worst case situation the safety and operation of the mainline itself may be
impaired by a backup onto the freeway. Therefore, special attention should be given to
providing sufficient capacity and storage for an at-grade intersection or a merge with the
crossing road. This may require adding addition lanes at the intersection or on the ramp
proper, or it could involve traffic signalization where the ramp traffic will be given priority.
The analysis must also consider the operational impacts of the traffic characteristics in either
direction on the intersecting road.

2. **Sight Distance.** Section 46-10.0 discusses the criteria for intersection sight distance. These
criteria also apply to a ramp/crossing road intersection. Special attention must be given to the
location of the bridge pier, abutment, sidewalk, bridge rail, roadside barrier, etc. These may
present major sight distance obstacles. The bridge obstruction and the required intersection
sight distance may result in the need to relocate the ramp/crossing road intersection.

3. **Wrong-Way Movements.** Wrong-way movements may originate at the ramp/crossing road
intersection. The intersection must be properly signed and designed to minimize the potential
for a wrong-way movement (e.g., channelization).
4. **Turn Lanes.** Additional turn lanes are often required at the end of ramp. Section 46-4.0 provides information on the design of turn lanes at intersections at-grade.

5. **Distance between Free-Flow Terminal and Structure.** The terminal of a ramp should not be near the grade-separation structure. If it is not practical to place the exit terminal in advance of the structure, the existing terminal on the far side of the structure should be well-removed. When leaving, drivers should be permitted some distance after passing the structure in which to see the turnout and begin the turnoff maneuver. Decision sight distance is recommended where practical. The distance between the structure and the approach nose at the ramp terminal should be sufficient for exiting drivers to leave the through lanes without undue hindrance to through traffic.

### 48-6.06 Access Control

Proper access control must be provided along the crossing road in the vicinity of the ramp/crossing road intersection or along a frontage road where present. This will ensure that the intersection has approximately the same degree of freedom and absence of conflict as the freeway itself. The access control criteria should be consistent with these goals.

Crossroads with interchange access typically provide the catalyst for development and traffic generators. To maintain the integrity of the freeway terminals it has become more apparent to extend access control beyond the ramp terminal nominal distance of 100-ft to 500-ft. Studies have shown that annual crash rates on cross roads with the first access located within 300 ft from the off ramp are at least 50% higher than those where the first access is located at the 300-750-ft range. At a minimum, the first access point should be located in urban areas 600-ft and 750-ft in rural locations from the off ramp away from the interchange along the cross road. On multi-lane crossroad terminals, access control should protect the distance to accommodate acceleration, weaving deceleration, transitioning, and storage to the first access point from the ramp terminal (See Figure 48-6D). In fully developed urban areas, these distances may not be achievable, but efforts should be made to avoid the minimum.

Figures 48-6E and 48-6F show access spacing based on one or more of four types of access connections upstream and downstream of an interchange terminus with both a two-lane crossroad and multi-lane crossroad configuration. Access connection types include:

1. Nearest access (all types)
2. Right-in/right-out
3. Un-signalized, full access
4. Signalized, full access
In addition, many areas have changed over the years from rural to urban. As indicated, INDOT has adopted different criteria for the access control at urban and rural interchanges. However, a change in area character alone is not a sufficient justification to alter the location of the full-access control line when an existing interchange will be rehabilitated or when INDOT receives requests for additional access points from outside interests.

Figure 48-6G show the extent of limited access right of way at ramp terminals. The figure states that, on the crossing road, the full-access control line should extend the indicated distance beyond “the ramp terminal.” For an exit ramp, this is defined as the tangent point of a radius return on the crossing road or the end of a taper for an entrance onto the crossing road (e.g., for an acceleration lane); i.e., the ramp terminal ends where the typical section of the crossing road resumes. A similar definition applies to ramp terminals for entrance ramps.
<table>
<thead>
<tr>
<th>Type of Intersecting Facility</th>
<th>Rural</th>
<th>Suburban</th>
<th>Urban</th>
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<td><img src="image" alt="Interchange Diagrams" /></td>
<td><img src="image" alt="Interchange Diagrams" /></td>
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<td><img src="image" alt="Interchange Diagrams" /></td>
<td><img src="image" alt="Interchange Diagrams" /></td>
</tr>
</tbody>
</table>

**FREEWAY INTERCHANGES**
(BASED ON FUNCTIONAL CLASSIFICATION OF INTERSECTING FACILITY)

Figure 48-2A
DIAMOND INTERCHANGE

Figure 48-2B
DIVERGING DIAMOND INTERCHANGE

Figure 48-2C
CONFLICT DIAGRAMS

CONFLICT POINT COMPARISON

<table>
<thead>
<tr>
<th></th>
<th>Crossing</th>
<th>Merging</th>
<th>Diverging</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Diamond</td>
<td>10</td>
<td>8</td>
<td>8</td>
<td>26</td>
</tr>
<tr>
<td>Diverging Diamond</td>
<td>2</td>
<td>6</td>
<td>6</td>
<td>14</td>
</tr>
</tbody>
</table>

Figure 48-2D
SINGLE POINT DIAMOND INTERCHANGE

Figure 48-2E
SINGLE ROUNDBOUT DIAMOND INTERCHANGE

Figure 48-2F
DOUBLE ROUNDBOUND DIAMOND INTERCHANGE

Figure 48-2G
THREE-LEVEL DIAMOND INTERCHANGE

Figure 48-2H
FULL CLOVERLEAF INTERCHANGE

Figure 48-2I
PARTIAL CLOVERLEAF (PARCLO)

Figure 48-2J (Page 1 of 2)
Major Road

ParClo - AB

ParClo - AB (Two Quad)

(E) Single Loop ParClo Interchange

PARTIAL CLOVERLEAF (PARCLO)

Figure 48-2J (Page 2 of 2)
THREE-LEG INTERCHANGE

Figure 48-2K
FULLY DIRECTIONAL INTERCHANGE

Figure 48-2L
(A) Semi-direct connection with multi-level structures

(B) Cloverleaf with Semi-direct connection

SEMI-DIRECTIONAL INTERCHANGE

Figure 48-2M
COORDINATION OF LANE BALANCE AND NUMBER OF LANES

Figure 48-3A
The recommended minimum length ($L$) is based on operational experience, need for flexibility, and adequate signing. Lengths should be checked in accordance with the procedure outlined in the Highway Capacity Manual (HCM). Refer to the HCM for the procedure to determine the length of the weaving section. The "$L$" distances noted above are measured between the painted noses (theoretical gore point). For EN-EN, a minimum distance of 300 ft is recommended between the end of the taper for the first entrance ramp and the painted nose for the succeeding entrance ramp.
(A) One-Sided Ramp Weave

(B) One-Sided Major Weave

(C) Two-Sided Weaving Section with Single Lane Ramps

(D) Two-Sided Weaving Section with Three Lane Changes

WEAVING SEGMENTS

Figure 48-3C
ALTERNATE METHODS OF DROPPING AUXILIARY LANES

Figure 48-3D
NOTES:
1. Point of controlling speed at ramp.
2. Shoulder transition from 10 ft to 8 ft.
3. See Figure 48-4K to determine applicable deceleration distance.
4. See Figure 48-4D for gore details.
5. Required length "L" above 800' shall be added to the length of the parallel lane segment.
6. For ramps on curves, see Section 48-5.06.

PARALLEL SINGLE LANE FREEWAY EXIT RAMP (PREFERRED) (SERVICE INTERCHANGE)

Figure 48-4A
NOTES:

1. Point of controlling speed at ramp.
2. Shoulder transition from 10 ft to 8 ft.
3. See Figure 48-4D for gore details.
4. Tapered ramp design should have a minimum controlling design speed complying with the middle range for ramp design speed as shown in Figure 48-5A.
5. Use of tapered single lane freeway exit ramp configuration requires approval from the Director of Highway Design and Technical Support.
6. For ramps on curves, see Section 48-5.06.

TAPERED SINGLE LANE FREEWAY EXIT RAMP
(SERVICE INTERCHANGE)

Figure 48-4B
NOTES:

1. Point controlling speed on the ramp.
2. "L" is the required acceleration length as shown in Figures 48-4H and 48-4I.
3. For entrance gore details see Figure 48-4D.
4. Transition pavement width from 18 ft to 12 ft.
5. Required additional length "L" above 600 ft minimum shall be added to the length of the parallel lane segment.
6. Where truck volumes are high use 70:1 taper.
7. Transition shoulder width from 8' to 10'.
8. For ramps on curves, see Section 48-5.06.

PARALLEL SINGLE LANE FREEWAY ENTRANCE RAMP (PREFERRED)
(SERVICE INTERCHANGE)

Figure 48-4C (Page 1 of 2)
NOTES:

1. Point of controlling speed on the ramp.

2. If the required "L" acceleration length is greater than 620 feet use a parallel single lane freeway entrance ramp (Figure 48-4C (page 1 of 2)).

3. For entrance gore details see Figure 48-4D.

4. For tapered design, the upstream design speed from the P.C.C. must meet the middle range design speed as shown in Figure 48-5A.

5. Transition shoulder width from 8 ft to 10 ft.

6. Transition pavement width from 18 ft to 12 ft.

7. Use of tapered single lane freeway entrance ramp configuration requires approval from the Director of Highway Design and Technical Support.

8. For ramps on curves, see Section 48-5.06.

TAPERED SINGLE LANE FREeways ENTRANCE RAMP
(SERVICE INTERCHANGE)

Figure 48-4C (Page 2 of 2)
Exit Gore

Curve Data:
-$\Delta$ = 3°00'00"
-$R$ = 3,819.72'
-$T$ = 100.02'
-$L$ = 200.00'
-$E$ = 1.31'

NOTE:
1. For ramps on curves, see Section 48-5.06.

Entrance Gore

Figure 48-4D (Page 1 of 2)
When the through lanes are not superelevated

When the thru lanes are superelevated and B is lower than A

When the through lanes are superelevated and B is higher than A

Ramp and through lane superelevated in same direction

Ramp and through lane superelevated in opposite direction

<table>
<thead>
<tr>
<th>SECTION A-A</th>
<th>SECTION B-B</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Diagram" /></td>
<td>Points c should be higher than point b.</td>
</tr>
<tr>
<td>Points b, c &amp; d should be progressively lower.</td>
<td>Points c should be equal to or lower than point b.</td>
</tr>
<tr>
<td><img src="image2.png" alt="Diagram" /></td>
<td>Points A, B, C &amp; D should be in same plane.</td>
</tr>
<tr>
<td>Points A, B, C &amp; D should be in same plane.</td>
<td>Points A, B, C &amp; D should be in same plane.</td>
</tr>
</tbody>
</table>

Note: Maximum rollover between mainline pavement and gore is 5%.

**GORE DETAILS**

Figure 48-4D (Page 2 of 2)
NOTES:

1. Point of controlling speed on the ramp.
2. See Figure 48-4D for entrance gore details.
3. Where truck volumes are high use 70:1 taper.
4. The minimum acceleration length "L" is 1000 ft. If the required acceleration length as shown in Figures 48-4H and 48-4I exceed 1000 ft., the additional length shall be added to the length of the parallel lane segment. This length may also be increased contingent on capacity of the downstream freeway segment.
5. For ramps on curves, see Section 48-5.06.

PARALLEL MULTI-LANE ENTRANCE RAMP
(SERVICE INTERCHANGE)

Figure 48-4E
NOTES:

1. Point of controlling speed at ramp.

2. See Figure 48-4B for exit gore details.

3. The designer must coordinate with the Traffic Engineering Corridor Development Office to verify these lengths.

4. For ramps on curves, see Section 48-5.06.

PARALLEL MULTI-LANE EXIT RAMP (SERVICE INTERCHANGE)

Figure 48-4F
TAPERED MULTI-LANE EXIT RAMP WITH OPTION LANE
(SERVICE INTERCHANGE)

Figure 48-4G
### Minimum Acceleration Lengths for Entrance Terminals

#### With Flat Grades of 3% or Less

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Speed Reached, V'a (mph)</th>
<th>Acceleration Length, L (ft) for Entrance Curve Design Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stop 15 20 25 30 35 40 45 50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>and Initial Speed, V'a (mph)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0 14 18 22 26 30 36 40 44</td>
</tr>
<tr>
<td>30</td>
<td>23</td>
<td>180 140 x x x x x x</td>
</tr>
<tr>
<td>35</td>
<td>27</td>
<td>280 220 160 x x x x x x x</td>
</tr>
<tr>
<td>40</td>
<td>31</td>
<td>360 300 270 210 120 x x x x</td>
</tr>
<tr>
<td>45</td>
<td>35</td>
<td>560 490 440 380 280 160 x x x x</td>
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<td>50</td>
<td>39</td>
<td>720 660 610 550 450 350 130 x x x</td>
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<td>43</td>
<td>960 900 810 780 670 550 320 x x x</td>
</tr>
<tr>
<td>60</td>
<td>47</td>
<td>1200 1140 1100 1020 910 800 550 420 180</td>
</tr>
<tr>
<td>65</td>
<td>50</td>
<td>1410 1350 1310 1220 1120 1000 770 600 370</td>
</tr>
<tr>
<td>70</td>
<td>53</td>
<td>1620 1560 1520 1420 1350 1230 1000 820 580</td>
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</table>

Source: 2011 AASHTO GDHS, Table 10-3

See Figure 48-4I per grade adjustments.

---

**Taper Type**

**Parallel Type**

**Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of 3% or Less**

---

**Figure 48-4H**
### Acceleration Lanes

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Ratio of Length on Grade to Length on Level for Design Speed (mph) of Last Ramp Curve</th>
<th>All Speeds</th>
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<tr>
<td></td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>3% ≤ Upgrade &lt; 4%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>1.30</td>
<td>1.30</td>
</tr>
<tr>
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<td>1.30</td>
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</tr>
<tr>
<td>70</td>
<td>1.50</td>
<td>1.60</td>
</tr>
<tr>
<td>4% ≤ Upgrade ≤ 6%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>1.50</td>
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### Deceleration Lanes

<table>
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<th>Ratio of Length on Grade to Length on Level for Design Speed of First Ramp Curve</th>
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<tr>
<td></td>
<td>&gt;3 to 4 % upgrade</td>
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<tr>
<td>All Speeds</td>
<td>0.9</td>
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<tr>
<td>All Speeds</td>
<td>&gt;4 to 6 % upgrade</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
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</table>

**NOTES:**
1. No adjustment is neeved for grades of flatter than 3%.
2. The grade in the table is the average grade measured over the distance for which the acceleration length applies.

**Example**

*Given:*
- Highway Design Speed: 70 mph
- Entrance Ramp Curve Design Speed: 45 mph
- Average Grade: 4.5% upgrade

*Problem:* Determine length of acceleration lane.

*Solution:* Figure 48-4H yields an acceleration length of 820 ft on the level. The grade adjustment for 45 mph is taken as the average of the values for 40 mph and 50 mph. According to the table shown above, this should be increased by the average of the increases shown for 40 mph (2.60) and 50 mph (3.00), or 2.80.

Therefore:

\[
L = (820 \text{ ft})(2.80) = 2300 \text{ ft}
\]

---

**GRADE ADJUSTMENT FOR ACCELERATION/DECELERATION (PASSENGER CAR)**

Figure 48-4I
LENGTHS FOR ACCELERATION 180 LB/HP TRUCK

Figure 48-4J
### Table 10-5

<table>
<thead>
<tr>
<th>Highway Design Speed (mph)</th>
<th>Speed Reached, $V_a$ (mph)</th>
<th>Deceleration Length, $L$ (ft)</th>
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Source: 2011 AASHTO GDHS, Table 10-5
See Figure 48-4I for grade adjustments

### Figures

**Parallel Type**

**Taper Type**

**MINIMUM DECELERATION LENGTH FOR EXIT TERMINALS WITH FLAT GRADES OF 3% OR LESS**

Figure 48-4K
MAJOR FORKS FOR SYSTEM INTERCHANGES
(TYPICAL SCHEMATICS)

Figure 48-4L
BRANCH CONNECTIONS FOR SYSTEM INTERCHANGES
(TYPICAL SCHEMATICS)

Figure 48-4M
**Design Element**

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<th>Section</th>
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<th>Ramps</th>
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<tr>
<td>Pavement Type (1)</td>
<td>Chapter 304</td>
<td>Asphalt/Concrete</td>
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</table>

**Shoulder (2)**

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<tr>
<th>Width</th>
<th>48-5.02</th>
<th>Multi-Lane</th>
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<tbody>
<tr>
<td>Usable: 9 ft</td>
<td>Paved: 8 ft</td>
<td>Usable: 11 ft</td>
</tr>
<tr>
<td>Usable: 5 ft</td>
<td>Paved: 4 ft</td>
<td>Usable: 5 ft</td>
</tr>
</tbody>
</table>

**Pavement Type (1)**

| Chapter 304 | Asphalt/Concrete |

**Cross Slope (3)**

| Travel Lane | 48-5.02 |
| Shoulder | 2% |
| Right: 4% Left: 2% |

**Superelevation**

| 48-5.03 | 4%, 6%, or 8% |

**Clear Zone Width**

| 49-2.0 | (4) |

**Side Slopes**

| Cut | Foreslope | 6:1 (5) |
| Ditch Width | 4 ft (6) |
| Backslope | 4:1 (7) |

| 6:1 to Clear Zone: 3:1 max to Toe |

---

(1) Pavement Type. The pavement selection will be determined by the Office of Pavement Engineering.

(2) Shoulder Width. Shoulder width criteria apply regardless of the presence of curb. Curbs should be considered only to facilitate drainage needs. Only sloping curb should be used.

(3) Cross Slope (Shoulders). For ramps on a curve, the shoulder cross slope will be the same as the traveled way. See Section 48-5.03.

(4) Clear Zone. This will vary according to design speed, traffic volume, side slopes, and horizontal curvature.

(5) Foreslope. See Section 49-2.0 and 49-3.0 for the lateral extent of the foreslope in a ditch section.

(6) Ditch Width. A "V" ditch should be used in a rock cut.

(7) Backslope. For an earth cut of 10 ft or deeper, the first horizontal 20 ft of the backslope will be sloped at a rate of 4:1. Then, a slope rate of 3:1 is normally used to the natural ground line. The backslope for a rock cut will vary according to the height of cut and the geotechnical requirements. See Sections 45-3.0 and 107-6.01.

**Highway Design, Speed, mph**

<table>
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<tr>
<th>40</th>
<th>45</th>
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<td>Upper Range, 85%</td>
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<td>Middle Range, 70%</td>
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<td>Lower Range, 50% (Loop Ramps Only)</td>
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<td>25</td>
<td>25</td>
<td>30</td>
<td>30</td>
<td>30</td>
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</table>

* Source: 2011 AASHTO GDHS, Table 10-1 Only highway design speeds of 50 mph or higher may be applied to a freeway or expressway exit.

**Ramp Design Speed**

**RAMP GEOMETRIC AND DESIGN CRITERIA**

**Figure 48-5A**
FIGURE 48-5B

SINGLE LANE RAMP TYPICAL SECTION

* The axis of rotation and PG may be moved to the centerline of the ramp to reduce transition length.
MULTI-LANE RAMP TYPICAL SECTION

Figure 48-5C
Left Side Freeway Lane Drop

End of Acceleration Lane 600' Min. 2000'-3000'

Right Side Freeway Lane Drop

End of Acceleration Lane 600' Min. 2000'-3000' 70:1 Min.

FREEWAY LANE DROP

Figure 48-6A
<table>
<thead>
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<th>Frontage Road Volume (vph)</th>
<th>Exit Ramp Volume (vph)</th>
<th>A (ft)</th>
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<td>200</td>
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<td>1240</td>
<td>850</td>
</tr>
<tr>
<td>2000</td>
<td>1380</td>
<td>970</td>
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</tbody>
</table>

Source: Transportation Research Record 682, Table 3

**NOTES:**

1. Table values are acceptable for planning purposes only. Final dimensions will be based on a detailed operational analysis. This design may only be used where necessary in a restricted urban area.

2. Total frontage road and exit ramp traffic volume between merge point to intersection with minor road.

3. Assumed to be 69% of total volume shown in first column.

4. Distance "A" is from the Exit Ramp center-line point of merge with the frontage road to the signalized intersection.

5. Distance "B" is determined on a case by case basis.

---

**RAMP AND CONTINUOUS FRONTAGE ROAD INTERSECTION**

(For Planning Only)

**Figure 48-6B**
Full Access Control Line

Limited Access R/W (LARW) and Access Control Line (ACL)

Frontage Road

Survey (Major Highway)

NOTES:

1. Full access control line should extend along the cross road beyond the ramp terminal extremity. See Figure 48-6D.

See Chapter 46 and Section 48-6.05 for Design Details at Intersections

Figure 48-6C

TYPICAL ACCESS CONTROL FOR A PARTIAL CLOVERLEAF INTERCHANGE
ACCESS CONTROL AT RAMP TERMINALS

Figure 48-6D
Minimum Spacing for Intersections and Commercial Entrances Near Interchange Area on Two-Lane Crossroads

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<th>Y</th>
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<td>600’</td>
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<tr>
<td>Rural</td>
<td>750’</td>
<td>1320’</td>
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X = Distance to first entrance on the right from end of off-ramp terminal.

Y = Distance to first four legged intersection measured from the end of the off-ramp terminal or from the start of the terminal for the on-ramp.

Z = Distance between the last entrance connection and the start of the terminal for the on-ramp.

Note: Spacing applies to both signalized and unsignalized intersections and commercial entrances regardless of the interchange configurations.

ACCESS CONTROL ON TWO-LANE CROSSROADS AT INTERCHANGES

Figure 48-6E
Minimum Spacing for Intersections and Commercial Entrances Near Interchange Areas on Multi-Lane Crossroads

<table>
<thead>
<tr>
<th>X or Z</th>
<th>Y</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>600'</td>
<td>1320'</td>
<td>1000'</td>
</tr>
</tbody>
</table>

M = Distance to the first directional median opening or no full median openings are allowed in non-traversable medians up to the first major intersection.

X = Distance to first entrance on the right from end of off-ramp terminal.

Y = Distance to first four legged intersection measured from the end of the off-ramp terminal or from the start of the terminal for the on-ramp.

Z = Distance between the last entrance connection and the start of the terminal for the on ramp.

Note: Spacing applies to both signalized and unsignalized intersections and commercial entrances regardless of the interchange configuration.

ACCESS CONTROL ON MULTI-LANE CROSSROADS AT INTERCHANGES

Figure 48-6F
NOTES:

1. Full access control line should extend along the cross road beyond the ramp terminal taper extremity (both sides of road) a minimum of 600 ft in urban areas and 750 ft in rural areas. The end of access control should be at opposite points, where feasible.

2. The auxiliary lane terminating the greater distance from the interchange area governs.

LIMITED ACCESS RIGHT OF WAY AT RAMP TERMINALS

Figure 48-6G
# CHAPTER 51

## Special Design Elements

NOTE: This chapter is currently being re-written and its content will be included in Chapter 307 in the future.

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The applicable design memorandum revision date is noted in brackets [ ] next to the affected section heading.
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CHAPTER 51

SPECIAL DESIGN ELEMENTS

51-1.0 ACCESSIBILITY [Rev Mar. 2016]

The *Americans with Disabilities Act* of 1990 (ADA) prohibits discrimination and ensures equal opportunity and access for persons with disabilities. Title II of the ADA prohibits discrimination on the basis of disability in the provision of services, programs, and activities by State and local governments. The Department, along with each local public agency, under ADA Title II, is required to provide ADA-compliant, otherwise known as accessible, facilities within the public right of way. Buildings within the public right of way, sidewalks, curb ramps, transit stops, on-street parking, parking lots, overpasses and underpasses are just a few examples of programs covered by Title II. Each private business which is considered to be a place of public accommodation, such as a retail business, restaurant, doctor’s office, law office, etc., is required under ADA Title III to provide an accessible facility on its private property.

The *2010 ADA Standards for Accessible Design* (2010 Standards) is the current standard for providing facilities that are readily accessible and usable by persons with disabilities. However, the guidelines were developed primarily for buildings and facilities outside the right of way. Pedestrian facilities within the public right of way contain elements to which the 2010 Standards cannot be readily applied. For this reason, the U.S. Access Board proposed guidelines specifically for pedestrian facilities in the public right of way - The *Public Rights-of-Way Accessibility Guidelines* (PROWAG). These guidelines are recommended as best practice by the Federal Highway Administration and are currently being evaluated as part of the federal rulemaking process. Once adopted as a regulation, with or without modifications, the guidelines will be mandatory.

The Department’s accessibility criteria meet the requirements of the ADA and seek to ensure that persons with disabilities may access the public right of way without discrimination. Unless otherwise noted, the Department’s accessibility criteria are based on the PROWAG, dated July 26, 2011. The applicable sections of the PROWAG are noted in brackets next to each section heading below. If local public agencies or local codes require standards which exceed the PROWAG, the stricter criteria should be used.
51-1.01 Transition Plan [Added Mar. 2016]

Under ADA Title II and Section 504 of the Rehabilitation Act of 1973, public agencies with more than 50 employees are required to complete a self-evaluation to identify services, policies and practices that are not accessible for persons with disabilities. A transition plan to correct the deficiencies is also required. The transition plan includes the following.

1. Identification of physical obstacles that limit the accessibility of facilities
2. Description of the methods to be used to make the facilities accessible
3. A schedule for implementing access modifications, and
4. Identification of the public official responsible for implementation of the transition plan

The transition plan must be updated and maintained until all barriers to accessibility are removed or documented to be technically infeasible to construct compliantly. See Section 40-8.04(01) Item 3 for submitting a determination of technical infeasibility or technical inquiry.

51-1.02 Pedestrian Access Route (PAR) [R302] [Added Mar. 2016]

A pedestrian access route or PAR is a continuous and unobstructed path of travel provided for pedestrians with disabilities within or coinciding with the pedestrian circulation path. The pedestrian circulation path is any prepared interior or exterior surface provided for pedestrian travel in the public right of way. Within the public right of way, the PAR typically includes sidewalks, pedestrian street crossings, and curb ramps, as well as overpasses and underpasses. Where the PAR is within a wider pedestrian circulation path, the accessibility criteria in this section apply only to the PAR.

The Department is responsible for ensuring the PAR is accessible within Department right of way. A business that serves the public and has a building with the building face on or nearly on the right of way or property line is responsible for ensuring that each building entrance or walk, etc., is accessible and compatible with the adjacent public right-of-way sidewalk.

51-1.03 Sidewalk [Rev. Mar. 2016]

A sidewalk provides a continuous path for pedestrians just as streets provide a continuous network to the motoring public. A sidewalk is part of a PAR and must meet the requirements of the ADA.
51-1.03(01) Location

The following should be considered when locating a sidewalk.

1. Sidewalk Continuity. Where a small section of the sidewalk must be rebuilt, for example to construct a compliant curb ramp, it is recommended to address the non-compliant aspects for the length of the sidewalk between logical termini. Logical termini may be the nearest intersection, drive, or other intersecting location.

2. Sidewalk Placement. Where new sidewalk is being considered, placement and setback along streets should take into account worn paths and buffer zones. A worn path where no sidewalk exists typically demonstrates the natural path pedestrians will take. Additional space should be provided for snow storage.

   The placement of a sidewalk should not require an exception to other Level One design criteria, such as shoulder or lane width.

3. Meandering sidewalks. Sidewalks that weave back and forth within the right-of-way are generally discouraged. While they may seem visually appealing, pedestrians prefer a direct, non-sinuous route. Meandering sidewalks may cause navigational difficulties for pedestrians with vision impairments.

4. Separation. It is desirable to provide a buffer space of 4 to 6 ft between the traveled way and the sidewalk. A buffer space provides for pedestrian comfort as well as facilitates installation of an accessible curb ramp.

   Where the speed limit of the adjacent roadway is 45 mph or less, a vertical curb should be used in conjunction with the sidewalk section to separate pedestrians from adjacent traffic.

   Where the speed limit of the adjacent roadway is greater than 45 mph, a barrier should be considered between the sidewalk and adjacent traffic if a sufficient separation cannot be provided.

5. Vertical drop off. Vertical drop offs are not addressed as part of the PROWAG. To address safety concerns, slopes adjacent to sidewalks should be as flat as practical. Consideration should be given to providing pedestrian railing where side slopes adjacent a sidewalk are 1:1 or steeper with a drop off greater than 24 in.
This section applies to sidewalks within the public right of way and meets the criteria described in the PROWAG, dated July 26, 2011. The applicable PROWAG sections are shown in brackets. A determination of technical infeasibility or technical inquiry must be approved for a sidewalk that does not meet the following criteria. See Section 40-8.04(01) Item 3 for submitting a request for determination of technical infeasibility or technical inquiry.

See INDOT Standard Drawings series 604-SWDK for sidewalk and driveway crossing details.

1. **Width [R302.3]**. The sidewalk width is measured exclusive of the curb, i.e. measured from the back face of curb. Sidewalks should be wide enough to accommodate the volume and type of pedestrian traffic expected.

   Where a sidewalk is used in conjunction with a buffer, the minimum width is 5 ft, exclusive of the curb. See Section 51-1.03(01) item 4, for desirable buffer widths. See Figure 51-1A, Sidewalk Clear Width.

   Where a sidewalk is located immediately adjacent the curb, a width of 6 ft should be used to allow additional space for street and highway hardware as well as to provide pedestrian comfort due to the proximity of traffic.

   Where the sidewalk serves commercial areas, schools, or other areas with concentrated pedestrian traffic, a width of 8 ft or greater may be appropriate.

   Where insufficient space is available, the sidewalk width may be reduced to 4 ft for short distances. Where the clear width is less than 5 ft, a passing space must be provided at no more than 200-ft intervals. The passing space must be a minimum of 5 ft by 5 ft. A taper rate of 6:1 should be used to widen and reduce the sidewalk width at passing spaces. See Figure 51-1B, Sidewalk Passing Space.

2. **Surface [R302.7]**. The sidewalk surface must be firm, stable, and slip-resistant. A change in level of up to 1/4 in. may be vertical and without edge treatment. A change in level of 1/4 in. to 1/2 in. must be beveled with a slope not greater than 1V:2H. A change in level of greater than 1/2 in. should be accommodated with a running slope in accordance with the curb ramp criteria.

   Where a grating is required within the PAR, the grating opening must not exceed 1/2 in. in the direction of pedestrian travel. Where a grating has elongated openings, the grating
must be placed so that the long dimension is perpendicular to the dominant direction of pedestrian travel.

3. **Cross Slope [R302.6]**. Cross slope is measured perpendicular to the direction of pedestrian travel. The maximum allowable cross slope of a sidewalk is 2.0%. A maximum cross slope of 1.5% is preferred and should be used as a design practice to reduce the likelihood of exceeding the maximum allowable cross slope during construction.

4. **Grade [R302.5]**. The grade or running slope is defined as the slope parallel to the direction of pedestrian travel. The grade of the sidewalk must not exceed the general grade established for the adjacent roadway.

5. **Protruding Object [R210]**. Protruding Objects such as street furniture, signal-controller cabinet, light standard, strain pole, utility pole, mailbox, sign support and other objects should not be placed within the width of the sidewalk. Protruding objects can be hazardous for pedestrians, especially pedestrians who are blind or have low vision. Where it is necessary to place a protruding object within the width of the sidewalk, a 4-ft minimum clear width may be provided for a short distance, see Figure 51-1A. For a shared-use path, protruding objects should not overhang into any portion of the shared-use path at or below 8 ft measured from the finished surface. This is to accommodate for bicycle traffic.

6. **Curb Ramp [R304]**. A curb ramp is used to lower or raise the sidewalk to connect with a public road approach. Each curb ramp must be in accordance with the criteria described in Section 51-1.04.

7. **Sidewalk Transition**. A sidewalk transition should be used as part of a sidewalk driveway crossing. The sidewalk transition is used to lower or raise the sidewalk to connect with a residential or commercial driveway without yield or stop control. A sidewalk transition has a maximum running slope of 8.33%. A maximum running slope of 8% is preferred as a design practice to reduce the likelihood of exceeding the maximum allowable running slope during construction. Sidewalk transition details are shown on the INDOT Standard Drawings series 604-SDWK.

8. **Sidewalk Driveway Crossing**. A sidewalk driveway crossing is where a sidewalk crosses a driveway with or without a sidewalk transition. Sidewalk driveway crossings should only be used at a residential or commercial driveway intersection without yield or stop control. Sidewalk driveway crossing details are shown on the INDOT Standard Drawings series 604-SDWK.
51-1.03(03) Sidewalk Plan Requirements [Added Mar. 2016]

Each sidewalk to be reconstructed should be detailed as follows:

1. **Plan Views.** Lines representing the sidewalk should be shown in plan view over existing survey or an aerial image. Use of an aerial image should consider the effect on file size.

2. **Spot Elevations.** In the absence of a full survey or paper relocation (PR) line, spot elevation at reasonable intervals must be included. Elevations at each side of the sidewalk, every 100 ft or break point should be tabulated or detailed.

3. **Dimensions.** Sections of sidewalk to be reconstructed should be shown with starting and ending stations. The width, especially where varying widths are expected, should be tabulated or detailed.

4. **Slopes.** Running slopes and cross slopes for each section of sidewalk should be tabulated or detailed. The preferred slopes should be used in design and shown on the plans.

For new construction, a compliant sidewalk can be detailed by calling out a standard sidewalk width, cross slope and running slope as part of a typical cross section. New construction assumes a new alignment or significant modification to an existing cross section and adequate right of way. Areas that fall outside the typical cross section (e.g. where the beginning and end of the project tie into an existing cross section) should be detailed as described for retrofits and reconstruction.

51-1.04 Curb Ramps [Rev. Mar. 2016]

Curb ramps provide access between the sidewalk and the roadway for wheelchair users. Note that although the design elements are similar, sidewalk curb ramp requirements are separate from the requirements for ramps that provide access in other locations outside the public rights of way, such as a ramp within or leading to a building, or a pedestrian overpass. Curb ramps for existing facilities which do not meet the PROWAG criteria must be included in the owner’s transition plan. See Figure 51-1C, Curb Ramp Components and Design Elements.

For project activities deemed as alterations in accordance with the Department of Justice/Department of Transportation Joint Technical Assistance on the Title II of the Americans with Disabilities Act guidance, ADA-compliant curb ramp installation or retrofit must be included within the scope of the project. See Figure 51-1D, Alteration vs. Maintenance activities.
51-1.04(01) Location [Rev. Mar. 2016]

Each curb ramp should be designed and placed to provide an unobstructed PAR while providing pedestrians the shortest but most direct route across a street.

In determining the location of a curb ramp, the designer should consider the following.

1. Where a raised sidewalk or improved surface intersects a public road approach, a curb ramp must be provided to transition to the elevation of the roadway.

2. Where sidewalk or other PAR continues on the opposite side of an intersection, opposing curb ramps must be provided.

3. Curb ramps should be located directly opposite one another for each pedestrian street crossing. Installing curb ramps in-line with the direction of pedestrian travel facilitates wayfinding for the blind and those with low vision.

4. Obstructions such as a signal controller box, planter, or signal pole base should be relocated away from the curb ramp wherever feasible. It is important that drivers be able to see the pedestrian using a curb ramp. Where it is not feasible to move the obstruction, the vehicle sight distance relative to the placement of the curb ramp should be considered.

The designer is responsible for identifying potential utility conflicts and mitigating conflicts to the extent feasible. If utilities are present, utility coordination should be in accordance with Chapter 104.

5. Crosswalk markings are preferred for all pedestrian street crossings and are required where a single curb ramp serves two directions of pedestrian traffic. See Figure 51-1L, Depressed Corner Curb Ramp. Where crosswalk markings are used, the full width of the ramp and clear space must be contained wholly within the markings. For placement of the crosswalk markings, see the Indiana Manual on Uniform Traffic Control Device (IMUTCD).

6. Stop line markings must not block the curb ramp or pedestrian street crossing, regardless of the use of crosswalk markings. The IMUTCD contains additional constraints on stop line markings.

7. The normal gutter flow line should be maintained through the curb ramp. Drainage structures should be placed as needed to intercept the flow prior to the curb ramp. Positive
drainage should be provided to carry water away from the intersection of the curb ramp and the gutter line, thus minimizing the depth of flow across the pedestrian street crossing.

51-1.04(02) Curb Ramp Components, Design Elements, and Design Criteria [Added Mar. 2016]

Curb ramp details are shown on the INDOT Standard Drawings series 604-SWCR. The details include the curb ramp components, design elements of each component, and the criteria for each design element. This information is summarized in Figure 51-1C, Curb Ramp Components and Design Elements.

Components and design elements are discussed below. The PROWAG section reference is shown in brackets adjacent to the description. Note that although the components are similar, curb ramp requirements are separate from the requirements for ramps that provide access in other locations outside the public right of way, such as in a building or at a pedestrian overpass.

An approved Determination of Technical Infeasibility or Inquiry must accompany each curb ramp that does not meet the PROWAG requirements. Examples of non-compliance include missing components, e.g. detectable warning surface or turning space, or a design element falling outside of the minimum or maximum criteria. See Section 40-8.04(01) Item 3 for requesting a Determination of Technical Infeasibility or Technical Inquiry.

Components

1.  **Ramp and Blended Transition.**  A ramp or blended transition is the component of a curb ramp that lowers the sidewalk or other pedestrian path to the roadway elevation.

2.  **Turning Space [R304.2.1 and R304.3.1].**  A turning space is a level area, running slope of 2.0% or less, critical for a wheelchair user to maneuver. A turning space must be provided at the top of a perpendicular curb ramp, the bottom of a parallel curb ramp, and where the PAR changes direction. It is acceptable for two perpendicular curb ramps to share a common turning space. A turning space is not required for a one-way directional curb ramp or a blended transition curb ramp.

   The minimum required clear dimensions of a turning space are 4 ft by 4 ft. Where the turning space is constrained by a curb, building, or other feature over 2 in at the back of the sidewalk, the minimum required clear dimensions are 4 ft by 5 ft, with the 5-ft dimension in the direction of the ramp running slope.
3. **Clear Space [R304.5.5]**. The clear space is provided beyond the grade break or detectable warning surface at the bottom of a ramp or blended transition to allow a wheelchair user to maneuver and align with the crosswalk markings. The minimum required clear dimensions are 4 ft by 4 ft. The clear space should be level and must be within the width of the pedestrian street crossing and wholly outside the parallel vehicle travel lane. The parallel vehicle travel lane is the lane where traffic is traveling parallel to the pedestrian street crossing. A grade break may fall within the clear space where the bottom of the ramp or blended transition meets the roadway pavement or gutter line; see item 9.

The clear space requires particular attention at diagonal ramps and other locations where the ramp is not in line with the direction of pedestrian travel.

4. **Flared Side and Returned Curb [R304.2.3]**. The flared side cannot be part of the PAR, but is part of the pedestrian circulation route. See Section 51-1.02. A flared side is required where the curb ramp intersects a sidewalk or other walkable surface. The maximum allowable slope of a flared side is 10%.

The returned curb may be used where the curb ramp intersects a buffer, sodded area, or other non-walkable surface or where protected from pedestrian travel by landscaping, street furniture, fencing, utility pole or railing.

5. **Detectable Warning Surfaces [R305]**. A detectable warning surface (DWS) warns visually-impaired pedestrians that they are entering the roadway. However, they are not intended to provide wayfinding.

The DWS consists of truncated domes aligned in a square or radial grid pattern and must extend the full width of the ramp, blended transition or median cut-through. Although PROWAG allows for a 2-in. border where forming is required, plans should show the DWS the entire width of the ramp. The need for forming is associated with the material selected for the DWS, and several materials are available on the Department’s Approved List for Detectable Warning Surfaces.

The DWS must contrast visually with the adjacent surfaces.

Each curb ramp and median cut through must contain a DWS except as follows.

a. Where a median cut through is less than 6 ft in the direction of pedestrian travel, DWS should not be placed. Where the median width is less than 6 ft, there is not
sufficient distance between surfaces to distinguish the boundary between pedestrian and vehicular routes.

b. Where a PAR intersects a residential driveway or a commercial driveway that does not contain stop or yield control, a DWS should not be placed. Where the PAR intersects a commercial driveway which contains stop or yield control, DWS should be provided.

The INDOT *Standard Drawings* series 604-SWCR contains DWS design elements and acceptable configurations based on the setback of the surface from the back of curb.

**Design Elements**

1. **Width [R304.5.1].** The minimum clear width of a ramp or blended transition is 4 ft. For a median cut through or median curb ramp, the minimum width is 5 ft. Where a curb ramp is used in conjunction with a shared-use path, it is preferred that the curb ramp width match the width of the shared-use path. See [Figure 51-C](#), Curb Ramp Components and Design Elements.

2. **Running Slope [R304.2.2 and R304.3.2 and R304.4.1].** The running slope of a ramp or blended transition is measured parallel to the direction of pedestrian travel. Providing the least slope possible is preferred. This will reduce the likelihood of exceeding the maximum allowable running slope during construction.

   a. **Ramp.** A ramp has a maximum running slope of 8.33%. A maximum running slope of 8% is preferred and should be used as a design practice to reduce the likelihood of exceeding the maximum allowable running slope during construction.

   b. **Blended Transition.** A blended transition has a maximum running slope of 5%. A maximum running slope of 4.5% is preferred and should be used as a design practice to reduce the likelihood of exceeding the maximum allowable running slope during construction.

   A running slope of 2% or less is considered level.

The running slope need not cause the ramp to exceed 15 ft in length. Where the ramp is “chasing the grade,” it may be terminated at the 15-ft length and a steeper grade used to tie back to the existing sidewalk. The running slope of the sidewalk outside of the 15-ft ramp
should not exceed the roadway profile grade plus 2% or should be ended at a logical termini location.

3. **Cross Slope [R304.5.3]**. Cross slope is measured perpendicular to the direction of pedestrian travel. The maximum allowable cross slope of a ramp, blended transition, turning space, or clear space is 2.0%. A maximum cross slope of 1.5% is preferred and should be used as a design practice to reduce the likelihood of exceeding the maximum allowable cross slope during construction.

   The cross slope may exceed 2.0% where it is acceptable for the pedestrian street crossing cross slope to exceed 2.0%. See Section 51-1.05 for pedestrian street crossing. See Figure 51-1E, Cross Slope at Pedestrian Street Crossing.

4. **Counter Slope [R304.5.4]**. The counter slope is a slope opposite to the general running slope of the ramp or sidewalk, typically the cross slope of the gutter or roadway at the foot of the ramp or blended transition. The counter slope must not exceed 5%. This maximum allows the rate of grade change not to exceed 13% when the maximum ramp running slope is used. Excessive rate of grade change compromises the ground clearance of a wheelchair footrest and may cause the wheelchair to tip.

   Where the rate-of-grade change exceeds 11%, a 2-ft level area should be provided adjacent the counter slope. See Figure 51-1F, Counter Slope and Rate of Grade Change.

5. **Grade Break [R304.5.2]**. The grade break at the top and bottom of a ramp must be perpendicular to the direction of the ramp running slope. This requirement is of particular importance where the curb is curved. It may be necessary at a corner with a larger radius to indent the grade break from the back of the curb to meet this requirement. Grade breaks are not permitted on the surface of the ramp.

   Where a curb is curved, the perpendicular curb ramp running slope meets the grade break at a right angle. On large radius corners, it will be necessary to indent the grade break on one side of the curb ramp in order for the curb ramp to meet the break at a right angle.

**51-1.04(03) Types of Sidewalk Curb Ramps [Rev. Mar. 2016]**

Details for placement of curb ramps and an illustration showing applicable locations for each curb ramp type are found on the INDOT *Standard Drawings* series 604-SWCR. Curb ramp design elements and criteria are discussed in Section 51-1.04(02).
Curb ramps are categorized by their orientation to the sidewalk or street.

1. **Perpendicular Curb Ramp.** Perpendicular curb ramps are the preferred design. A perpendicular curb ramp has a running slope that cuts through or is built up to the curb at right angles and serves a single direction of pedestrian traffic. See Figure 51-1G, Perpendicular Curb Ramp.
   
   a. **Components.** Perpendicular curb ramps include a single ramp and may have flared sides or returned curbs. A turning space is required at the top of the ramp. A clear space is required at the bottom of the ramp. Detectable warning surfaces are required. Crosswalk markings are preferred.
   
   b. **Selection Considerations.** A distance of 10 - 12 feet from the back of curb to the back of sidewalk is necessary to accommodate a perpendicular curb ramp assuming it is adjacent to a 6 in. curb.

   Taller curb height, a constraint at the back of sidewalk, and a running slope less than 8.33% will all increase the total lengths required between the curb and back of sidewalk.

Where an existing sidewalk cannot be widened to accommodate a perpendicular or tiered perpendicular curb ramp, a parallel curb ramp should be considered. A tiered perpendicular curb ramp consists of lowering the sidewalk prior to the curb ramp using sidewalk transitions. See Figure 51-1G, Perpendicular and Tiered Perpendicular Curb Ramps.

2. **Parallel Curb Ramp.** Parallel curb ramps are a preferred design. A parallel curb ramp has a running slope that is in line with the direction of sidewalk travel. The ramps lower the sidewalk to a turning space where a turn is made to enter the pedestrian street crossing. See Figure 51-1H, Parallele Curb Ramps.
   
   a. **Components.** Parallel curb ramps include two ramps and typically do not have flared sides or returned curbs. A turning space is required at the bottom of the ramps. A curb may be required at the back edge of the ramp to retain soil or delineate a building or other constraint adjacent to the ramp. A clear space is required at the bottom of the turning space if placed at an intersection. A midblock crossing does not require a clear space. Detectable warning surfaces are required. Crosswalk markings are preferred.
   
   b. **Selection Considerations.** A parallel ramp is typically used at a midblock crossing, and requires that the sidewalk be at least 4 ft wide. For narrow sidewalks at an
intersection, paired parallel curb ramps can be placed. A paired parallel curb ramp should not be installed where it is possible to install paired perpendicular curb ramps.

3. **Median Pedestrian Crossing**  A median pedestrian crossing consists of a raised median at the intersection or midblock location to separate pedestrians from traffic. See Figure 51-1I, Median Pedestrian Crossings.

   a. Components. A median pedestrian crossing can be cut through at street level or consist of a series of perpendicular curb ramps. The cut-through configuration can provide useful cues to the direction of travel. Detectable warning surfaces are required within a median that has a width greater than or equal to 6 ft. Crosswalk markings are preferred.

4. **Blended Transition Curb Ramp**. A blended transition curb ramp is a connection between the level of the pedestrian walkway or sidewalk and the level of the pedestrian street crossing that has a running slope of 5% or less. A blended transition curb ramp serves more than one direction of pedestrian traffic. See Figure 51-1J, Blended Transition Curb Ramp.

   a. Components. A blended transition curb ramp includes a single blended transition and may have flared sides. A turning space is not required behind the blended transition, however, where the blended transition running slope exceeds 2.00%, a 4-ft minimum sidewalk should continue behind the blended transition. Detectable warning surfaces are required where the blended transition is flush with the pedestrian street crossing. Crosswalk markings are required.

   b. Selection Considerations. A blended transition curb ramp is suitable for a range of sidewalk conditions, however they provide limited directionality for visually impaired users. Safety considerations need to be evaluated for possible increased interaction with turning vehicles.

5. **One-Way Directional Curb Ramp**. A one-way directional ramp is a single perpendicular curb ramp that serves a single direction of pedestrian traffic. There is no change in direction at the top or bottom of these ramps. See Figure 51-1K, One-Way Directional Perpendicular Curb Ramp.

   a. Components. The components of a perpendicular curb ramp apply except that a turning space is not required at the top of the ramp. Crosswalk markings are preferred.
b. Selection Considerations. A one-way directional ramp may be specified only at a corner where the PAR continues across a single intersecting roadway with no change in direction.

6. **Depressed Corner Curb Ramp.** A depressed corner curb ramp is a single parallel curb ramp that serves two directions of pedestrian traffic. See Figure 51-1L, Depressed Corner Curb Ramp.

   a. Components. The components of a parallel curb ramp apply. Crosswalk markings are preferred.

   b. Selection Considerations. A depressed corner curb ramp is suitable for a range of sidewalk conditions, however, they provide limited directionality for visually impaired users. Safety considerations need to be evaluated for possible increased interaction with turning vehicles.

7. **Diagonal Curb Ramp.** A diagonal curb ramp is a single perpendicular curb ramp located at the apex of the corner at an intersection, and serves two directions of pedestrian traffic. See Figure 51-1M, Diagonal Curb Ramps.

   a. Components. The components of a perpendicular ramp apply. Crosswalk markings are required.

   b. Selection Considerations. A diagonal curb ramp should not be specified for new construction. Although diagonal curb ramps may save construction costs, they create potential safety hazards and mobility problems for pedestrians including reduced maneuverability and increased interaction with turning vehicles. For alterations where existing physical constraints prevent paired curb ramps from being installed at an intersection, a diagonal ramp may be specified. Each diagonal curb ramp, excluding flared sides and the clear space at the bottom of the ramp, must be wholly contained within the crosswalk markings and outside the parallel vehicle travel lane. Where both the turning space and clear space cannot be provided, a diagonal ramp is not appropriate for the site.


Each curb ramp to be retrofit into an existing facility (e.g. a sidewalk that does not have a curb ramp) or reconstructed (e.g. an existing non-compliant curb ramp) should be detailed as follows:
1. **Plan Views.** Lines representing the curb ramp and DWS should be shown in plan view over existing survey or an aerial image. Use of an aerial image should consider the effect on file size.

2. **Spot Elevations.** Elevations at each side of the top and bottom of the ramp, turning space, and flared side should be tabulated or detailed.

3. **Dimensions.** Lengths and widths for each ramp, turning space, DWS, flared side, and return curb should be tabulated or detailed.

4. **Slopes.** Running slopes and cross slopes for each ramp, turning space, and flared side should be tabulated or detailed. The preferred slopes should be used in design and shown on the plans.

For new construction, a curb ramp should be detailed using a detail drawing, table, or a combination of the two. The detail drawing or table should include the curb ramp location, and required curb ramp components and design element criteria, e.g. width, length, cross slope, running slope and flared side slope.

New construction assumes a new alignment or significant modification to an existing cross section and adequate right of way. Areas that fall outside the typical cross section, e.g. where the beginning and end of the project tie into an existing cross section, should be detailed as described for retrofits and reconstruction.

Where a new or reconstructed curb ramp is located on a corner with an existing pedestrian pushbutton assembly, access to the pushbutton assembly must be perpetuated. See Section 51-1.06 for pedestrian pushbutton assembly placement and configuration.

An approved Determination of Technical Infeasibility or Technical Inquiry must accompany each curb ramp that does not meet the ADA requirements. Examples of non-compliance include missing components, e.g. DWS or turning space, or a design element falling outside of the minimum or maximum criteria. See Section 40-8.04(01) Item 3 for requesting a Determination of Technical Infeasibility or Technical Inquiry.

### 51-1.05 Pedestrian Street Crossing [R302.5 and R302.6] [Added Mar. 2016]

The pedestrian street crossing is the continuation of the PAR across a roadway.

The cross slope of the pedestrian street crossing is the same as the profile grade of the roadway through the crossing. The maximum allowable cross slope of a pedestrian street crossing is as follows.
1. Where the pedestrian street crossing contains yield or stop control, e.g. a yield sign or stop sign, the maximum cross slope is 2%.

2. Where the pedestrian street crossing does not contain yield or stop control, e.g. signalized, the maximum cross slope is 5%.

3. Where the pedestrian street crossing is located at a midblock crossing, the maximum cross slope is the roadway profile grade. See Figure 51-1E. Cross Slope at Pedestrian Street Crossing.

The grade of the pedestrian street crossing is the same as the cross slope of the roadway through the crossing. The maximum allowable grade of a pedestrian street crossing is 5%.


This section applies to both accessible pedestrian signal (APS) and non-APS pedestrian pushbutton assemblies unless otherwise stated. See Section 502-3.04(05), Pedestrian Signal for additional information.

51-1.06(01) Accessible Pedestrian Signal [R209 and R307] [Rev. Nov. 2016]

An accessible pedestrian signal (APS) is a device that communicate information about the WALK and DON’T WALK intervals at signalized intersections in visual and non-visual format. This device is essential for a pedestrian who is blind or has low vision to effectively navigate the crossing.

For a new signal installation, signal modernization, or intersection improvement project, the Department will determine whether pedestrian heads are appropriate for the location. If pedestrian heads are appropriate, an APS Study in accordance with Section 502-3.04(05) must be conducted.

51-1.06(02) Placement and Configuration [Rev. Nov. 2016]

The placement and configuration of the pedestrian pushbutton assembly is critical to proper function. Engineering judgment is required to determine the optimal installation at each crossing. Variations in curb radius, available right of way, presence of a buffer or curb ramp, and existing infrastructure make each crossing unique.
Details for pedestrian pushbutton assembly placement and configuration are shown on RPD 805-T-201d until such time as they are incorporated into INDOT Standard Drawings series 805-PPBA. The details are in accordance with the IMUTCD 4E.08 – 4E.13 and the PROWAG.

1. **Pushbutton Clear Space.** [R404] A pushbutton clear space must be provided adjacent a pedestrian pushbutton assembly. The minimum required clear dimensions are 4 ft by 4 ft. The clear space must be free of grade breaks, may overlap a curb ramp turning space or sidewalk, and may overlap a ramp with a running slope of 2% or less. Providing a clear space that is concurrent with the curb ramp turning space is preferred. This approach increases the likelihood that the dimensional and slope requirements will be met and provides a reasonable distance to the crosswalk.

   The running slope and cross slope of a pushbutton clear space are based on the orientation of the pushbutton assembly. See Figure 51-1P, Pushbutton Clear Space. The running slope may be consistent with the grade of the sidewalk. The cross slope must be 2.00% maximum.

2. **Placement.** Where the offset between the face of curb or edge of pavement and the back edge of sidewalk is 10 ft or less, placing the pedestrian pushbutton assembly outside the back edge of sidewalk, is preferred. Where the assembly can be accessed from two directions, consideration should be given to centering the assembly relative to the crosswalk. That is, do not require a pedestrian to travel down one ramp, then up another to reach the assembly.

   Where the offset between the face of curb or edge of pavement and the back edge of sidewalk is greater than 10 ft, or other site constraints exist, e.g. a building at the back edge of sidewalk, placement within the sidewalk or buffer may be necessary.

   a. **Pedestrian Pushbutton Assembly Outside the Back Edge of Sidewalk.** A pedestrian pushbutton assembly should not be placed more than 5 ft outside the associate crosswalk. A pushbutton assembly should be centered adjacent a pushbutton clear space. See Figure 51-1Q, Pedestrian Pushbutton Assembly Outside the Back Edge of Sidewalk, Preferred.

      A pushbutton assembly must not be blocked by obstructions, e.g. behind guardrail.
b. **Pedestrian Pushbutton Assembly Within a Sidewalk or Buffer.** A pedestrian pushbutton assembly should not be placed more than 5 ft outside the associate crosswalk. A pushbutton assembly should be adjacent a pushbutton clear space. Centering on the pushbutton clear space is not required, however the grade break guidance in Item 3 would apply.

The distance from the nearest face of a pushbutton assembly to face of the curb or edge of pavement should be between 1.5 ft and 6 ft and should not be greater than 10 ft. A minimum offset of 1.5 ft from the face of curb or edge of pavement will allow a wheelchair user to remain out of traffic while actuating the pushbutton assembly. A minimum offset of 1.5 ft also provides an appurtenances-free zone along the roadway. See Section 55-5.02, Item 5.

A 4-ft minimum sidewalk clear width must be provided where a pushbutton assembly is placed within a sidewalk.

See Figure 51-1R, Pedestrian Pushbutton Assembly Within a Sidewalk or Buffer.

A pushbutton assembly must not be blocked by an obstruction, e.g. behind street furniture.

3. **Grade Break.** Where a grade break is adjacent a pushbutton clear space it is preferred to offset the nearest face of the pedestrian pushbutton assembly a minimum of 1.5 ft from the grade break. A wheelchair user positioned on a grade break may become unstable while actuating the pushbutton assembly and enter into traffic prematurely. Figure 51-1R, Pedestrian Pushbutton Assembly Within a Sidewalk or Buffer.

4. **Spacing.** Where two pedestrian pushbutton assemblies are provided on the same corner of a signalized intersection or within a median, the pushbutton assemblies should be separated by at least 10 ft. Where constraints prevent a 10-ft separation, pushbutton assemblies may be placed closer together or on the same pole. Where two APS pushbutton assemblies are closer than 10 ft., special features must be included in accordance with IMUTCD 4E.10 and RSP 805-T-201, Accessible Pedestrian Signals until such time as the RSP is incorporated into the Standard Specifications. RSP 805-T-202, Accessible Pedestrian Signals with Speech Walk Messages should be completed by the designer and included in the contract, when an APS pushbutton assembly is required.

Where a median cut through is less than 6 ft in the direction of pedestrian travel and the pedestrian street crossing is signalized, the signal should be timed for a complete crossing of the street.
5. **Mounting Height and Side Reach.** [R406] The actuator of the pedestrian pushbutton assembly must be located between 42 in. and 48 in. above the pushbutton clear space and within a 10-in. unobstructed side reach. See Figure 51-1S, Pedestrian Pushbutton Assembly Mounting Height and Side Reach. Where pole placement is limited, a 6 in. or 12 in. pushbutton assembly extension may be used to meet the side reach criteria.

6. **Actuator.** The actuator must have a 2-in. minimum diameter and contrast visually with the housing or mounting. See Section 502-3.03(05), Detector.

7. **Orientation.** The face of a pedestrian pushbutton assembly must be aligned parallel to the direction of pedestrian travel on the associated crosswalk. See Figure 51-1T, Orientation of Pedestrian Pushbutton Assembly.

8. **Signage.** Pedestrian signal signs must be mounted immediately above or incorporated into the pedestrian pushbutton assembly.

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Each pedestrian pushbutton assembly should be detailed as follows:

1. **Plan Views.** A symbol and lines representing the pushbutton assembly and pushbutton clear space, respectfully, should be shown in plan view over existing survey or an aerial survey. Use of an aerial survey should consider the effect on file size.

2. **Stations and Offsets.** The station and offset for each pushbutton assembly should be tabulated or detailed. Where two pedestrian pushbutton assemblies are provided on the same corner of a signalized intersection or within a median, the distance between the two should also be tabulated or detailed.

3. **Dimensions.** Length and width for each pushbutton clear space should be tabulated or detailed.

4. **Slopes.** Slopes of the pushbutton clear space, if not detailed with the curb ramp or sidewalk, should be tabulated or detailed.

5. **Pushbutton Mounting Height.** Where an existing pushbutton mounting height requires adjustment to meet ADA criteria, the required adjustment should be noted as a callout on the plans. Otherwise, “No mounting height adjustment required” should be noted as a callout
for the existing pushbutton. For new pushbutton installation, Standard Drawing series E 805-PBBA will govern and a mounting height note is not required on the plans.

6. Pushbutton Side Reach. Where a new or existing pushbutton side reach requires an extension to meet ADA criteria, the extension length should be noted as a callout on the plans. Otherwise, “No side reach adjustment required” should be noted as a callout.

7. Pushbutton Direction. The predominant direction of pedestrian traffic crossing serviced by the pushbutton should be noted as a callout on the plans (e.g. N-S or E-W).

An approved Determination of Technical Infeasibility or Technical Inquiry must accompany each pushbutton assembly or pushbutton clear space that does not meet the ADA requirements. Examples of non-compliance include a pushbutton assembly placement or pushbutton clear space slope or dimensions falling outside of the minimum or maximum criteria. See Section 40-8.04(01) Item 3 for requesting a Determination of Technical Infeasibility or Technical Inquiry.

51-1.07 Transit Stops and Transit Shelters [R213 and R308] [Rev. Mar. 2016]

Locating transit stops at signalized intersections is recommended to increase usability for pedestrians with disabilities. Where transit stops or transit shelters are provided, the following will apply.

1. **Transit Stop.** A new transit stop may be constructed at sidewalk or street level. Where transit stops serve vehicles with more than one car, accessible boarding and exiting areas must be provided for each car. Boarding and exiting area criteria are as follows.
   
   a. **Dimensions.** The minimum clear length is 8 ft, measured perpendicularly from the curb or roadway edge. The minimum clear width is 5 ft, measured parallel to the roadway.

   b. **Grade.** The grade of boarding and exiting area parallel to the roadway must match the roadway grade to the extent practical. The grade perpendicular to the roadway must not exceed 2%. A maximum grade of 1.5% is preferred and should be used as a design practice to reduce the likelihood of exceeding the maximum allowable cross slope during construction

   c. **Connection.** Boarding and exiting areas must be connected to the street, sidewalk, or pedestrian path by a PAR.
2. **Transit Shelter.** Where a new or replacement transit shelter is provided, it must be connected by a PAR to a boarding and exiting area. The transit shelter may be located within or outside of the boarding area. However, the shelter must not reduce the width of the PAR to less than 4 ft. Clear space requirements must be in accordance with PROWAG section R404.

3. **Signage.** Each new transit-route identification sign should be sized based on the maximum dimensions permitted by federal, State, or local regulations or ordinances.

**51-1.08 On-Street Parking [R214 and R309] [Rev. Mar. 2016]**

Where on-street parking is marked or metered, the on-street parking design should be in accordance with the accessibility criteria as follows.

1. **Minimum Number.** Figure 51-1N, Minimum Number of Accessible Spaces, provides the criteria for the minimum number of on-street accessibility spaces.

2. **Location.** On-street accessible parking spaces should be dispersed throughout the project area. Accessible parking spaces should be located where the street has the least crown and grade and close to key destinations. The sidewalk adjacent to a parallel parking space should be free of signs, street furniture and other obstructions to allow for vehicle side-lift or ramp operation.

3. **Parallel Parking Adjacent Wide Sidewalk.** A minimum parking space width of 8 ft with an access aisle of 5-ft width should be provided where the width of the adjacent sidewalk is 14 ft or greater. The travel lane should not encroach into the access aisle. Figure 51-1 O, Accessible On-Street Parking, illustrates the parking configuration.

4. **Parallel Parking Adjacent Narrow Sidewalk.** A minimum parking space width of 8 ft should be provided. An access aisle is not required. When an access aisle is not provided, the accessible parking space should be located at the end of the block face.

5. **Perpendicular or Angled Parking.** A minimum parking space width of 8 ft with an access aisle of 8.0 ft should be provided at street level the full length of the parking space. Two parking spaces are allowed to share a common access aisle.

6. **Signage.** Each accessible parking space must be identified by the international symbol of access. The sign requirements are contained in the *Manual of Uniform Traffic Control*.
Devices (MUTCD). For parallel parking spaces, the signs must be placed at either the head or the foot of the parking space. Signs must not obstruct the PAR.

7. **Curb Ramp.** A curb ramp in accordance with Section 51-1.04(02) must connect the access aisle to the PAR. The curb ramp should not be located within the area of the access aisle. A parking space adjacent to an intersection may be served by the sidewalk curb ramp at the intersection, provided that the path of travel from the access aisle to the sidewalk curb ramp is within the pedestrian street crossing area.

8. **Parking Meter.** At an accessible parking space, the parking meter must be located at the head or foot of the parking space so that there is no interference with the operation of a vehicle side-lift or a passenger-side transfer. The parking meter must not obstruct the PAR.

A part of an accessible route with a running slope steeper than 5% should be considered a ramp and must be in accordance with the PROWAG. This includes providing handrails. These requirements do not apply to sidewalks or curb ramps within the public rights of way. The following criteria apply to a ramp on an accessible route.

1. **Running Slope.** The running slope must be between 5% and 8.33%; however, the flattest possible slope should be used.

2. **Cross Slope and Surface.** The cross slope of a ramp surface must not exceed 2.0%. The ramp surface must be in accordance with the sidewalk-surface criteria described in Section 51-1.03(02).

3. **Width.** A width of 5 ft is recommended to facilitate maintenance and snow removal for outdoor conditions. The minimum clear width of a ramp is 3 ft. Where handrail is installed, the minimum clear width must be provided and the ramp or landing width extended 1.0 ft beyond the inside face of the handrail. This extension prevents wheelchair casters and crutches tips from slipping off the ramp surface.

4. **Rise.** The rise for any ramp must not exceed 2.5 ft.

5. **Landing.** A ramp must have a level landing at the bottom and top of each ramp. A landing must be in accordance with the following.

   a. The width must be at least as wide as the widest ramp leading to it.
b. The clear length must be a minimum of 5 ft.

c. Where the ramp changes direction at a landing, the minimum required dimensions are 5 ft by 5 ft.

d. Slopes must not exceed 2% in any direction.

6. **Handrail [409]**. If a ramp has a rise greater than 6 in. or a horizontal projection greater than 6 ft, it must have handrails on both sides. A handrail is not required for a curb ramp or sidewalk within public rights of way. A handrail must be in accordance with the following.

a. Handrails should be provided along both sides of a ramp segment. The inside handrail on a switchback or dogleg ramp must be continuous.

b. If a handrail is not continuous, it must extend at least 1 ft beyond the top and bottom of the ramp and be parallel with the floor or ground surface.

c. The clear space between the handrail and the wall must be 1.5 in.

d. The gripping surface must be continuous along the top and side. The bottom of the handrail gripping surface must not be obstructed for more than 20% of the length. The gripping surface and any surface adjacent to it must be smooth and free of sharp or abrasive elements.

e. The top of the gripping surface must be mounted between 34 in. and 38 in. above the ramp surface.

f. The end must be either rounded or returned smoothly to the floor, wall, or post.

g. A handrail must not rotate within its fittings.

7. **Edge Protection**. A ramp or landing with a drop-off must have a curb, wall, railing, or projecting surface that prevents wheelchair casters and crutch tips from slipping off the ramp. A curb must be of minimum height of 2 in. A barrier must prevent passage of a 4-in. diameter sphere where any portion is within 4 in. of the ground surface.

8. **Outdoor Conditions**. An outdoor ramp and its approaches must be designed so that water will not accumulate on the walking surface.
51-1.10 Stairway [R408 ][Rev. Mar. 2016]

A stairway must not be part of a PAR, but may be part of the larger pedestrian circulation path. See Section 51-1.02.

Where a stairway is provided within a building or as part of an access route to a building or facility, it must be accessible. Components include treads, tread surface, risers, nosing and handrails.

Where handrails are provided, they must be in accordance with PROWAG Section 409.

51-1.11 Building [Rev. Mar. 2016]

For interior accessibility criteria, the following will apply:

1. **New.** Each new building, airport terminal, rest area, weigh station, or transit station (e.g., station for rapid rail, light rail, commuter rail, intercity bus, intercity rail, high-speed rail, or other fixed guideway systems) must meet the accessibility criteria set forth in the 2010 ADA Standards for Accessible Design (2010 Standards). The designer should review the 2010 Standards to determine the appropriate accessibility requirements for the building interior, including rest rooms, drinking fountains, elevators, telephones and other facility features.

2. **Existing.** For alterations made to an existing building or facility, the design must meet the accessibility requirements to the maximum extent feasible. The designer should review the 2010 ADAAG to determine the appropriate criteria.

51-2.0 REST AREA

A rest area, information center, or scenic overlook is functional and desirable element of the complete highway development and is provided for the safety and convenience of the highway user. Many have been constructed along freeways and other major arterials. The location and design of a rest area is based on individual highway facility and site needs. The need for a new rest area will be determined by the Office of Environmental Services in conjunction with the district office.
**51-2.01 Location**

A rest area may be located on a freeway or other major arterial. Along a freeway, two are usually paired together (i.e., one on each side of the freeway). At a State line, only one rest area or welcome center for the incoming traffic may be provided. The following provides additional information in determining the need and location of a rest area.

**51-2.01(01) Spacing on an Interstate Route**

The recommended average spacing of rest areas is approximately one hour of driving time or 50 to 60 mi. It may be desirable to provide closer spacing for special conditions (e.g., scenic view, information center). Local conditions may warrant spacing which is greater than 50 to 60 mi (e.g., through a major metropolitan area).

**51-2.01(02) Site Considerations**

Once it has been determined that a rest area is required and the general area has been selected, the actual location of the rest area is selected based upon the following considerations.

1. **Appeal.** A rest area is a showplace for out-of-State visitors. If practical, it should be placed to take advantage of natural features (e.g., lakes, scenic views, points of special or historic interest).

2. **Welcome Center.** It is desirable to locate this facility close to a State line. This location provides the opportunity to personally present information on the State along with local attractions. A rest area located well within the State may only provide information racks for literature distribution.

3. **Geometrics.** The site should be located away from any other interference, such as an interchange or a bridge. The rest-area entrance should desirably be at least 3 miles from the nearest interchange.

4. **Environmental Considerations.** The site should be located or designed so that surface runoff or treatment-plant discharges will not adversely affect streams, lakes, wetlands, etc.

5. **Median.** A rest area should not be located in a median unless it can be serviced via a left-hand exit and entrance.
6. **Size.** The rest area should be large enough to provide sufficient parking capacity, needed facilities, picnic and stretch areas, and to retain existing landscaping features.

7. **Right of Way.** Right-of-way costs should be factored into the location decision. To allow for future expansion, a 40-year design life should be considered based on a straight-line traffic projection.

8. **Topography.** A rest area should be located where the natural topography is favorable to its development.

9. **Development.** A rest area should not be placed adjacent to or near an area which has been zoned as residential.

10. **Emergency.** The location choice should consider the proximity to emergency services.

11. **Water and Sewer.** The rest area should have an adequate water supply. Water availability should be determined during the site selection process prior to the development of plans. If a commercial sanitary-treatment plant is unavailable, the site must be large enough to provide for adequate sewage-treatment facilities. Recreational-vehicle dumping facilities may be provided.

12. **Other Utilities.** Other utilities, such as telephone and electricity, should always be provided.

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**51-2.02 Design**

**51-2.02(01) Exit and Entrance**

The access to and from the rest area should be designed in accordance with Section 48-4.0. Reverse curves should not be used. If deemed necessary, they should be designed in accordance with Section 43-3.07. Full-depth shoulders should be provided along both exit and entrance ramps to the ramp extremities (i.e., the ends of the ramp tapers).

Adequate signing and pavement markings must be provided. These traffic-control devices should be placed in accordance with Part VII, the INDOT Standard Drawings, and the MUTCD.

**51-2.02(02) Buffer Separation**
The separation between the rest area facility and the highway mainline should be wide enough to discourage individuals from stopping on the mainline and crossing over to the facility. At a minimum, a 35-ft buffer area should be provided between the mainline pavement and parking areas. A buffer separation of 175 ft or more is preferable. Fencing should be provided in the buffer area between the ramps and should desirably be located beyond the mainline clear zone.

51-2.02(03) Rest-Area Usage

Predicting rest area usage is the key factor in determining the location and sizing of a rest area. The designer must first determine the proportion of mainline traffic that will be using the rest area. This determination is dependent upon rest-area spacing, trip length, rest-area location, time of year, traffic composition, highway classification, etc. The designer should use data from nearby or similar rest areas to estimate the expected traffic entering the rest area. In the absence of historical data, Figure 51-2A, Design Guide for Rest-Area Facility (Interstate Route or Freeway), and the following may be used.

1. **Design Year.** The design year for traffic projection should be 20 years.

2. **Highway Characteristics.** A rest area on a highway that passes through recreational or historic areas tends to have fewer trucks and a higher percentage of passenger cars and RVs with trailers. Where the general purpose of the highway is to move commercial traffic between cities, a rest area tends to have a higher truck usage.

3. **Trip Length.** On a highway where the trip length is typically less than 100 mi (e.g., between two major cities), there is a significant reduction in the proportion of the passing traffic using the facility.

4. **Temporal Factors.** In a recreational area, rest-area usage is the highest during a summer weekend. During the day, passenger cars tend to make up a higher percentage of the rest-area usage. At night, trucks and RVs tend to make up the higher percentage of rest-area usage.

51-2.02(04) Parking

Rest-area parking capacity depends upon the type of usage expected for the rest area. Figure 51-2A, Design Guide for Freeway Rest-Area Facility, provides the formula and other factors to consider when determining the appropriate design hourly volume for passenger cars, passenger cars
with trailers, and trucks. Consideration should be given to adding additional truck parking spaces if the rest area is located close to major delivery or distribution centers.

Parking areas for passenger cars and trucks should be separated from each other within the rest area. This should be accomplished by providing separate parking areas on opposite sides of the building. However, a separator (e.g., curbing) or pavement markings may be used in a restrictive location. Figure 45-1B illustrates typical parking designs for a passenger car. Angular parking is preferred to parallel parking because it requires less time to enter and exit.

Figure 51-2B illustrates a typical angle-parking design for a truck or recreational vehicle. The design vehicle for angular truck parking is the WB-20 vehicle.

51-2.02(05) Pavement Design

Pavements for exit and entrance ramps, truck parking area, and truck connector roadway should be designed using a 14-in. portland cement concrete pavement on 3 in. of coarse aggregate No. 8 on 6 in. of compacted aggregate No. 53. The pavement area to be used only by passenger cars may be designed using a 10-in. portland cement concrete pavement on 3 in. of coarse aggregate No. 8 on 6 in. of compacted aggregate No. 53.
51-2.02(06) Cross Slopes

All ramps and connector routes should have a 2% cross slope. Parking areas typically should be designed with a 2% cross slope. A 5% maximum grade may be used. If practical, handicapped parking areas should not exceed 1%.

51-2.02(07) Facilities

A rest area provides a building with rest rooms and public information services, picnic tables and shelters, benches, sidewalks, drinking fountains, and trash collectors. It may also include vending machines, provided the machines are accessible from outside the building. The designer should ensure that sufficient facilities are available to accommodate the expected usage of the rest area. Figure 51-2A, Design Guide for Freeway Rest-Area Facility, provides the recommended total number of comfort facilities. Figure 51-2C, Guidelines for Comfort Facilities, should be used to determine the recommended number and types of fixtures. Dual men’s and women’s facilities (minimum of 2 each) should be provided to allow for cleaning, maintenance, etc. The total number of fixtures should be divided equally between the rest rooms. If practical, the designer should also consider providing exclusive unisex rest rooms for handicapped individuals. The building should be adequately sized to provide 120 ft² of floor area for each sanitary facility plus an additional 200 ft² of floor space. The rest-area building must be in accordance with all Indiana Department of Fire Prevention and Public Safety building codes.

51-2.02(08) Utilities

Where permanent facilities are provided, an adequate drinking-water supply, a wastewater disposal system, and a power supply will be required. These are required to bring the facilities into accordance with federal, Indiana Administrative Code (IAC), and Indiana Department of Environmental Management (IDEM) regulations, and local ordinances. Where practical, connection to existing wastewater treatment facilities and drinking-water supplies is the most desirable option.

A dedicated drinking-water treatment system will require a security system, ozone addition for deposition of iron, chlorine treatment, phosphate treatment, and backflow prevention to prevent contamination of the stored water and the water from the well. The drinking-water treatment system structure should be placed at least 4 ft horizontally clear of other structures. For a purchased-water system, automated chlorine testing and addition will also be required. Drinking-water treatment should otherwise be in accordance with IAC 327.
A dedicated wastewater disposal system will require a testing laboratory. Wastewater treatment units will require protection from exposure to direct sunlight, covers, or other means that prevent animals, bird feces, or external debris from entering the system, and shelter or other means that keeps the wastewater temperature within a specified range. A standby electric generator, surge control tank with dissolved oxygen sensor, trash collection tank, fixed film media filters, sand filters, ultraviolet disinfection, diffusers, and a splitter box are also required. The wastewater disposal system trash collection tank should be placed upstream of the surge control tank. Wastewater treatment should otherwise be in accordance with IAC 327 and 329.

A remote telemetry system will be required for the drinking-water and wastewater treatment facilities, lift stations, and locations where the water is purchased.

As a minimum, the telemetry system should include the following:

1. A portable laptop computer for data access and system interaction, including an operator training manual.

2. The computer software should be compatible with and be able to enter data onto IDEM’s report forms. The forms are accessible through IDEM’s website, at http://www.in.gov/idem/5157.htm#waterforms.

3. The interaction shall include an alarm to alert the plant operator (when the operator is both on-site and off-site) when the system’s conditions are not within the required parameter limits.


5. The interaction shall include the ability to automatically add treatment chemicals.

The designer should develop appropriate specifications and call for appropriate pay items for this additional work. The specifications should comply with the Ten State Standards requirements. The Office of Environmental Services’ Environmental Policy Team will review and approve the specifications.

The IDEM is responsible for approval of the final wastewater treatment and drinking-water supply options.

Telephones are usually also included. Proper lighting provides the patron an added sense of security and safety. Section 502-4.0 provides additional information on lighting design.
51-2.02(09) Landscaping

The rest area should be landscaped to take advantage of existing natural features and vegetation (see Section 51-8.0). Paths, sidewalks, and architectural style should fit naturally into the existing surroundings. The designer should coordinate the landscaping plan with the Services and Cultural Resources Team. A chain link fence should be placed between the parking areas and the adjacent roadway to enhance pedestrian safety.

51-2.02(10) Accessibility for the Handicapped

A rest area must be designed to properly accommodate physically handicapped individuals; including grounds, picnic areas, ramps to picnic areas, buildings, automatic door openers, sidewalk ramps, and signage. The designer must realize that an accessible route is required between the truck and RV parking area to the rest-area facilities. Section 51-1.0 provides the handicapped accessibility criteria for exterior features within a rest area. The ADA Accessibility Guidelines for Buildings and Facilities provides the handicapped-accessibility criteria for interior features.

51-3.0 WEIGH STATION

A truck weigh station installation is used to weigh trucks, to provide for vehicular safety inspection, or to provide a source of data for planning and research. The determination of the need for a truck weigh station is a combined effort of INDOT, the Indiana State Police, the Department of Revenue, and the Bureau of Motor Vehicles.

51-3.01 Location

Indiana has adopted the Point-of-Entry concept for locating a new weigh station. A weigh station is to be located only at or near a State line for inbound trucks on an Interstate route.

The actual selection of a truck weigh station site is controlled by right of way limitations and by geometric and topographic features (i.e., at the crest of a hill). It is desirable to select a site in a location where there is adequate right of way and where geometric, topographic, or environmental features lend themselves to the most economical development without undue site preparation and expense. The possibility of truck traffic circumventing the facility is also considered in locating the site of the weigh station.
51-3.02 Design

Figure 51-3A illustrates a typical truck weigh-station layout. In addition, the following should be considered.

1. **Exit and Entrance Junctions.** Desirably, the exit and entrance should be designed for large trucks. Section 48-4.0 provides design criteria for these elements, including truck acceleration and deceleration lengths.

2. **Exit and Entrance Ramps.** The minimum paved width is 28 ft, including a 4-ft left shoulder and 8-ft right shoulder. The shoulders should be designed with a full-depth pavement structure along both exit and entrance ramps to the ramp extremities (i.e., the ends of the ramp tapers). The cross slope will typically be 2% for the entire width, including shoulders.

3. **Pavement Design.** Pavements for ramps and the scale area should be designed using a 14-in. portland cement concrete pavement on 4 in. of coarse aggregate No. 8 on 3 in. of compacted aggregate No. 53. The parking area should have 12 in. portland cement concrete pavement on 4 in. of coarse aggregate No. 8 on 3 in. of compacted aggregate No. 53.

4. **Geometrics.** The weigh station area should be designed so that backing maneuvers are not required (e.g., pull-through parking). All pavement geometrics should be designed to accommodate off-tracking for a WB-20 design vehicle (Indiana Design Vehicle).

5. **Maximum Grade.** A short upgrade of as much as 5% does not unduly interfere with truck or bus operations. Consequently, for new construction it is desirable to limit the maximum grade to 5%. The grades across a weigh-in-motion scale must be 0% for 100 ft before and after the weigh-in-motion scale.

6. **Buffer Separation.** There should be a 30-ft minimum buffer strip between the weigh station facility and the mainline pavement. A wider separation is desirable.

7. **Storage Length for Scale.** There should be sufficient space to queue trucks waiting for the scale without backing up onto the mainline. This distance will be based on the number of trucks on the mainline, length of trucks, expected hours of operation, and time required for actual weighing. For design considerations, the design vehicle can be assumed to be the WB-20 truck. With the rapid advance in research on scales (e.g., weigh-in-motion), the designer should check with other Department entities or other agencies to determine the most appropriate time factor.
8. **Safety Inspection.** A weigh station will also be used by the Indiana State Police as a safety-inspection station. Therefore, a separate inspection building will be required. This building should be designed to accommodate a total of two WB-20 design vehicles, one in each of the adjacent bays.

9. **Violation Storage.** A space should be provided to store trucks that are either overweight or which have failed the safety inspection. These areas should be designed to accommodate the WB-20 design vehicle. Figure 51-2A, Design Guide for Freeway Rest-Area Facility, provides the design criteria for a WB-20 angular truck storage area.

10. **Traffic Control Devices.** Adequate signing and pavement markings should be provided prior to and at the truck weigh station. These traffic control devices should be designed and placed in accordance with the MUTCD and the INDOT Standard Drawings. The designer should contact the Production Management Division’s Traffic Design Team for information regarding design for an electronic “Open / Closed” sign. Special signing will also be necessary for the internal traffic flow through the weigh station, such as at the weigh-control area and the inspection building.

11. **Lighting.** Section 502-4.0 provides information on lighting design.

12. **Inspection Building.** An inspection building should be designed for year-round use with sufficient space for computer operations, a service counter for permit issuances, and an emergency shower facility for hazardous-material removal. The inspection building should be in accordance with all local building codes and OSHA criteria.

13. **Hazardous Materials.** A 1600-gal. tank is required on site for the storage of hazardous materials from leaking or overflowing trucks. A detention basin with flow-release controls is required to contain surface runoff from the parking area.

14. **Landscaping.** The weigh station should be designed to minimize the effect on existing vegetation. The designer should also ensure that any new or existing plants will not affect the driver’s sight distance to the weigh station or any critical point within the weigh station. Section 51-8.0 provides additional information on the Department’s landscaping policy.

**51-4.0 OFF-STREET PARKING**

A proposed highway project may incorporate some form of off-street parking. Typical applications may include the following:
1. providing off-street parking to replace on-street parking which will be removed as part of a proposed project;

2. the construction of a park-and-ride lot for commuters; or

3. the construction of a new rest area or improvement to an existing rest area.

The following provides criteria specifically for an off-street parking lot. Section 51-2.0 discusses that for a rest area.

### 51-4.01 Location of Park-and-Ride Lot

The Office of Environmental Services, in conjunction with the district office, determines the location of a park-and-ride lot during the planning stage. However, the designer usually has some control over the best placement of the lot when considering layout details, entrance and exit locations, and traffic flow patterns.

A park-and-ride lot should be located at a strategic point where transfers can conveniently be made from auto to carpooling or transit modes. Considerations that will affect the location of the parking facility are as follows.

1. **Location.** The lot should be convenient to residential areas, bus and rail transit routes, and the major highways used by commuters.

2. **Congestion.** The location should precede any points of congestion on the major commuting highway to maximize its benefits.

3. **Connections.** There should be sufficient capacity on connections between the lot and the major commuting highway.

4. **Design.** The site location must be compatible with the design and construction of the lot. The designer should consider property costs, terrain, drainage, sub-grade soil conditions, and available space in relation to the required lot size, visibility, and access.

5. **Land Use.** The location of the lot should be consistent with the present and future adjacent land use. Visual and other impacts on surrounding areas should be considered. Where necessary, site sizing and design should allow for buffer landscaping to minimize the visual impact.
6. **Size.** The lot must be large enough to accommodate its expected usage. Studies by the Office of Environmental Services will determine the size of the lot and will determine the number of bus-loading areas.

### 51-4.02 Layout

The following should be considered when laying out a park-and-ride facility.

1. **Entrances and Exits.** Entrances and exits should be located to have the least disruption to existing traffic (e.g., away from intersections) and still provide the maximum storage space. A combined entrance and exit should preferably be as close to mid-block as practical. Where entrances and exits are separated, the entrance should be on the upstream side of the traffic flow nearest the lot and the exit on the downstream side. There should be at least one exit and entrance for each 500 spaces in a lot.

   Each entrance or exit should be designed as a commercial drive according to the design criteria described in Chapter 46. The typical design vehicle will be a BUS or SU.

2. **Drop-off and Pick-up Zone.** Drop-off and pick-up zones for buses and autos should be clearly separated from each other and from the parking area to avoid as many internal traffic conflicts as possible. The bus loading and unloading zone should be serviced by the innermost parking lanes. Therefore this zone should be adjacent to the terminal loading and unloading area. Handicapped parking and the separate kiss-and-ride area should be serviced by the next closest parking lane. The number of parking spaces for a drop-off zone is between 20 and 60.

3. **Traffic Circulation.** Traffic circulation should be arranged to provide maximum visibility and minimum conflict between small vehicles (autos and taxis) and large vehicles (large vans and buses). Also, adequate maneuvering room must be provided for larger vehicles. A counterclockwise circulation of one-way traffic is preferred. This allows vehicles to unload from the right side.

4. **Pedestrian and Bicyclist Considerations.** The designer should consider pedestrian and bicycle routes when laying out a park-and-ride lot. Entrance and exit points in an area with high pedestrian volume should be avoided, if practical. Sidewalks should be provided between the parking area and the modal transfer points.

   Crosswalks should be provided where necessary and clearly marked and signed. In a high-volume lot, fencing may be warranted to channel pedestrians to appropriate crossing points.
A crossing at a major two-way traffic circulation lane should have a refuge island separating the travel directions.

A bicycle parking area should be provided with stalls that allow the use of locking devices. If a large volume of bicycle traffic is expected, a designated bicycle lane to and from the bicycle parking area should be provided.

5. **Accessibility.** Parking must be readily accessible and usable by persons with disabilities in accordance with the *2010 ADA Standards for Accessible Design*. Considerations include but are not limited to minimum number of accessible spaces, location of accessible spaces, van-accessible spaces, and the presence of curb ramps to access loading zones.

### 51-4.03 Design Elements

The following elements should be considered in the design of a park-and-ride facility.

1. **Parking-Stall Dimensions.** Parking-stall dimensions vary with the angle at which the stall is arranged relative to the aisle. Figure 51-4A, Parking-Stall Dimensions, provides the design dimensions for a 9 ft x 18 ft parking stall based on one-way circulation and angle parking. The typical stall width (measured perpendicular to the vehicle when parked) ranges from 8.5 ft to 9.5 ft. The recommended minimum stall width for self-parking of long-term duration is 8.5 ft. For higher-turnover self-parking, a stall width of 9 ft is recommended. Stall width at a supermarket or other similar parking facility, where large packages are prevalent, should desirably be 9.5 ft to 10 ft.

2. **Bus Loading Area.** A bus loading and unloading area should be designed to provide for continuous counterclockwise circulation and for curb parking without backing maneuvers. The traffic lanes and the curb loading area should each be 12-ft wide. Figure 51-4B provides criteria for the recommended length of a bus loading area.

3. **Sidewalk Dimensions.** The sidewalk should be at least 6 ft wide. In a loading area, the width should be at least 12 ft. The accessibility criteria for the handicapped must be met for a new lot (see Section 51-1.0).

4. **Cross Slope.** To provide proper drainage, the minimum cross slope on the parking lot should be 2%. The maximum, cross slope should not exceed 5%.

The lot should be designed directing runoff into existing drainage systems. If water impoundment cannot be avoided along a pedestrian route, bicycle route, or standing area,
drop inlets and underground drainage should be provided. In a parking area, drainage should be designed to avoid standing water. Part IV provides additional information for the proper hydraulic design of drainage elements.

5. **Pavement.** A typical pavement design for the parking area is 3 in. of hot asphaltic concrete on 6 in. of aggregate base. For a bus route, the minimum pavement section should be 3½ in. of hot asphaltic concrete on 10 in. of aggregate base. For additional information on pavement design, see Chapter 52.

6. **Lighting.** The lot should be lighted for pedestrian safety and lot security. Section 502-4.0 provides information on lighting design.

7. **Shelter.** A pedestrian shelter is desirable if loading areas for buses or trains are provided. The shelter should provide approximately 5.5 ft² of covered area per person. As a minimum, the shelter should provide lighting, benches, and trash receptacles. Routing information signs and a telephone should also be considered. For handicapped-accessibility requirements, see Section 51-1.0.

8. **Fencing.** The need for fencing around a parking lot will be determined as required.

9. **Signs.** Signs should be provided to direct drivers and pedestrians to appropriate loading zones, parking areas, bicycle facilities, handicapped parking, or entrances and exits.

10. **Landscaping.** Landscaping may be provided to minimize the visual impact of the parking lot by providing a buffer zone around the perimeter of the lot or to improve the aesthetics of the lot itself. Space should be provided for a 10-ft to 20-ft buffer zone around the lot to accommodate vegetation screens. Traffic islands and parking-lot separators provide suitable locations for shrubs and trees. Landscaping should include low-maintenance vegetation which does not cause visibility or security problems. For information on appropriate vegetation selections, the designer should contact the Services and Cultural Resources Team.

**51-4.04 Maintenance Considerations**

Maintenance should be considered in the design, including the following.

1. A 10-ft to 20-ft snow shelf should be provided around the perimeter of the lot, at least on two sides, to provide storage space during snow removal. This area can coincide with the
buffer zone around the lot, provided that the entire area is not filled with shrubs or trees. Fencing should be placed outside the snow shelf.

2. Raised traffic islands should be kept to a minimum. Raised corrugated islands are preferred.

**51-5.0 BUS STOP AND BUS TURNOUT**

**51-5.01 Location**

**51-5.01(01) Bus Stop**

If local bus routes are located on an urban or suburban highway, the designer should consider their impact on normal traffic operations. The stop-and-go pattern of local buses will disrupt traffic flow, but certain measures can minimize this disruption. The location of a bus stop is particularly important. It is determined not only by convenience to patrons but also by the design and operational characteristics of the highway and the roadside environment. If the bus must make a left turn, for example, a bus stop should not be located in the block preceding the left turn. Common bus-stop locations are shown in Figure 51-5A, On-Street Bus Stop.

Some considerations in selecting an appropriate bus-stop location are as follows.

1. **Far-Side Stop.** The far side of an at-grade intersection is superior to a near-side or mid-block bus stop. A far-side stop produces fewer impediments to through and right-turning traffic, it does not interfere as much with intersection sight distance, and it lends itself better to a bus turnout.

2. **Mid-Block Stop.** A mid-block bus stop may be advantageous where the distance between intersections is large or where there is a fairly heavy and continuous transit demand throughout the block. It may be desirable if there is a high bus-stop demand located at mid-block. A mid-block bus stop may also be considered if right turns at an intersection are frequent (250 in peak hour) and a far-side stop is not practical.

3. **Near-Side Stop.** A near-side stop allows easier vehicle re-entry into the traffic stream where curb parking is allowed. At an intersection where there is a high volume of right-turning vehicles, a near-side stop can result in traffic conflicts and should be avoided. However, a near-side stop should be used where the bus will make a right turn at the intersection.
51-1.01(02) Bus Turnout

Interference between buses and other traffic can be reduced significantly by providing a bus turnout. A turnout helps remove stopped buses from the through lanes and provide a well-defined user area for a bus stop. A turnout should be considered under the following conditions.

1. The street provides arterial service with high traffic speeds and volumes and high-volume bus patronage.

2. Right-of-way width is sufficient to prevent adverse impact on sidewalk pedestrian movements.

3. Curb parking is permitted but is prohibited during peak hours.

4. There are at least 500 vehicles per hour in the curb lane during peak-hour traffic.

5. Bus volume does not justify an exclusive bus lane, but there are at least 100 buses per day and at least 10 to 15 buses during the peak hour.

6. The average bus dwell time exceeds 10 s per stop.

7. At a location where specially-equipped buses are used to load and unload handicapped individuals.

51-5.01(03) Selection

The Office of Environmental Services, in conjunction with the district office and the local transit agency, will determine the location of a bus stop or bus turnout. However, the designer usually has some control over the best placement of a bus stop or turnout location when considering layout details, intersection design, and traffic-flow patterns.

51-5.02 Design

51-5.02(01) Bus Stop

Figure 51-5A provides the recommended distance for the prohibition of on-street parking near a bus stop. Where articulated buses are expected to use a stop, an additional 20 ft should be added to this distance. An additional 50 ft of length should be provided for each additional bus expected to
stop simultaneously at any given bus-stop area. This allows for the length of the extra bus (40 ft) plus 6 ft between buses. Changes in parking restrictions will require Official Action by INDOT.

**51-5.02(02) Bus Turnout**

The following design criteria will apply.

1. The desirable width is 12 ft, and the minimum width is 10 ft.

2. The full-width area of the turnout should be at least 50 ft long. Where articulated buses are expected, the turnout should be 70 ft. For a two-bus turnout, add 50 ft.

3. Figure 51-5B illustrates the design details for a bus turnout. In the transition areas, an entering taper not sharper than 5:1 and an exit taper not sharper than 3:1 should be provided. As an alternative, a horizontal curve of 100 ft radius may be used on the entry end and a horizontal curve of 50 ft to 100 ft radius may be used on the exit end. When a turnout is located at a far-side or near-side location, the cross-street area can be assumed to fulfill the need for the entry or exit area, whichever applies.

**51-5.02(03) Bus-Stop Pad**

Each new bus stop which is constructed for use with lifts or ramps must be in accordance with the accessibility criteria set forth in Section 51-1.0.

**51-5.02(04) Shelter**

The need for a bus-stop shelter will be determined by the Office of Environmental Services in conjunction with the local transit agency. The designer should consider the following in the design of a shelter.

1. **Visibility.** To enhance passenger safety, the shelter sides should provide maximum transparency as practical. The shelter should not be placed such that it limits the general public’s view of the shelter interior.

2. **Selection.** The local transit agency should be contacted to determine if it uses a standardized shelter design.
3. **Appearance.** The shelter should be pleasing and blend with its surroundings. The shelter should also be clearly identified with transit-company logo symbols.

4. **Accessibility.** A new shelter must be designed to be in accordance with the accessibility criteria set forth in Section 51-1.07.

5. **Placement.** The shelter should not be placed where it will restrict vehicular sight distance, pedestrian flow, or handicapped accessibility. It should also be placed so that waste and debris are not able to accumulate around the shelter.

6. **Responsibility.** The local transit agency is responsible for providing and maintaining the shelter.

7. **Capacity.** The maximum shelter size is based upon the maximum expected passenger accumulation at a bus stop between bus runs. The designer can assume approximately 5.5 ft\(^2\) per person to determine the appropriate shelter size. See Section 51-1.07 for minimum accessibility requirements.

### 51-6.0 RECREATIONAL ROAD

Recreational-road design criteria are applicable to a road on a scenic drive or a Department of Natural Resources property such as a State park or other recreational area. The objective for this type of facility is to provide a safe highway and still retain the aesthetic, ecological, environmental, and cultural amenities of the area.

#### 51-6.01 Functional Classification

A recreational road is functionally classified as a primary access road, circulation road, or area road. A primary access road provides access between a general-public-use highway and the recreational facility. A circulation road provides for the movement between activity sites within the recreational facility. An area road allows for the direct access to individual activity areas such as a campground, park area, boat-launching ramp, picnic area, scenic overlook, or historic site. Figure 51-6A illustrates a typical recreational-road functional-classification network.
51-6.02 Design

Strict adherence to highway criteria for this type of road is usually inappropriate and unwarranted. Design speed is usually low and driver expectancy is such that the reduction of design criteria does not produce serious safety concerns. Therefore, the designer should use engineering judgment to ensure that the design criteria fit the terrain and expected usage of the highway. Figure 51-6B provides the recommended geometric design criteria for a recreational road. However, for a primary access road which is a part of the county or state highway system, the geometric design criteria as described in Chapter 53 or 55 for the appropriate functional classification should be used. In addition to Figure 51-6B, the designer should consider the following.

51-6.02(01) Design Vehicle

Depending on the nature of the recreational area, the most common design vehicle may be a passenger car, passenger car with a travel trailer, passenger car with a boat trailer, motor home, a motor home with a boat trailer, or possibly a bus. Where garbage pickup or other maintenance vehicles are required, an SU may be the most appropriate design vehicle.

The selected design vehicle should be used to determine lane widths, vertical clearances, intersection design, etc.

51-6.02(02) Stopping Sight Distance

Figure 51-6B provides the minimum stopping sight distances for a 2-lane or a 1-lane road. On a 2-directional 1-lane road, sufficient sight distance must be provided to allow one vehicle to reach a turnout or for both vehicles to stop before colliding. This distance is considered to be twice the stopping sight distance.

51-6.02(03) Vertical Alignment

Figure 51-6B provides the recommended $K$ values for vertical curves, maximum grades, and vertical clearances. Chapter 44 provides additional information on vertical-alignment design.
51-6.02(04) Horizontal Alignment

Straight tangent sections are often aesthetically undesirable and often physically impractical. Figure 51-6B provides the recommended minimum radius based on an $e_{max}$ of 4%. However, on a primary access road, an $e_{max}$ of 6% may be used. For a design speed of 20 mph or lower, superelevation is often unnecessary and impractical. Chapter 43 provides additional information on horizontal alignment for a paved roadway. An unpaved roadway is not superelevated.

For a narrow roadway with minimum radii, it may be necessary to provide travelway widening on the inside of a sharp curve. AASHTO A Policy on Geometric Design of Highways and Streets provides information for the design of pavement widening. The design vehicle for pavement widening will be the motor home with a boat trailer (MH/B).

51-6.02(05) Cross Section

Figure 51-6B provides the recommended cross-section widths for travel lanes, shoulders, and auxiliary lanes. The use of wider pavements is often aesthetically objectionable and often unwarranted. The designer must balance the safety benefits of a wider roadway with those of aesthetic and environmental concerns.

Where traffic volumes are less than 100 vehicles per day, it may be feasible to use a 2-directional, 1-lane roadway. This roadway type is often desirable from an economic and environmental standpoint. Where a 1-lane roadway with 2-directional traffic is used, turnouts for passing should be provided. Traffic convenience requires that such turnouts be intervisible, provided on each blind curves, and supplemented as necessary so that the maximum distance between turnouts is not more than 1000 ft. A turnout should be a minimum of 10 ft wide for a length of 50 ft and should have a 30-ft taper on each end. For an extra-long or extra-wide vehicle, these dimensions may need to be adjusted.

On a primary access road, the foreslopes and backslopes should be 4:1 or flatter. However, on a circulation or area road this criterion is often aesthetically undesirable. At a lower speed, steep slopes typically do not present a problem. However, maintenance operations may be better facilitated by the use of flatter slopes. The ditch section, typically a V ditch, should be deep enough to satisfactorily accommodate the expected design flow and provide for satisfactory drainage of the pavement base and sub base.
51-6.02(06) Roadside Safety

On a primary access road, an obstruction-free zone of 10 ft should be provided from the edge of the travel lane. However, use of a smaller width is appropriate where economic or environmental concerns dictate. The use of an obstruction-free zone on a circulation or area road is less critical due to its lower speed and traffic volume. Nevertheless, the designer should provide as wide an obstruction-free zone as practical where the accident potential is greater than normal (e.g., a sharp horizontal curve at the end of a long, steep downgrade). Section 55-5.0 provides additional information on the application of obstruction-free zone.

Roadside barriers should only be installed at points of unusual danger. Where barriers are installed, they should blend in naturally with the surrounding environment (e.g., wood rails on wood posts). For information on acceptable roadside barriers along a recreational road, the designer should contact the Indiana Department of Natural Resources’ Engineering Division.

51-7.0 NONMOTORIZED-VEHICLE-USE FACILITY

This Section provides a source of guidance to implement the Indiana Trails, Greenways, and Bikeways plan. A safe, convenient, and well-designed facility is essential to encourage public use. This Section provides information on the development of facilities to enhance and encourage safe nonmotorized-vehicle, pedestrian, and bicycle travel. The majority of bicycling takes place on ordinary roads with no dedicated space for a bicyclist. A bicyclist can expect to ride on most roadways, as well as separated shared-use paths or sidewalks, where permitted, if conditions warrant.

This Section provides information to accommodate bicycle and pedestrian shared-use traffic in most environments. It provides guidelines that are valuable in attaining a facility design that is sensitive to the needs of the bicyclist or other user.

This Section should be used in conjunction with the remainder of this Manual, the Manual on Uniform Traffic Control Devices (MUTCD), and the AASHTO Guide for the Development of Bicycle Facilities.

This Section is consistent with, and similar to, typical engineering practices which are described elsewhere in this Manual and the MUTCD.
51-7.01 Definitions

The following definitions will apply.

1. **Barrier.** A containment device used to separate a shared-use path from an adjacent roadway where motor-vehicle speeds are high. A pedestrian/bicycle-type barrier is appropriate for placement along a facility where motor vehicles are not present.

2. **Bicycle.** Every vehicle propelled solely by human power upon which a person can ride, having two tandem wheels, except for a scooter or similar device. The term also includes a three- or four-wheeled human-powered vehicle, but not a tricycle for a child.

3. **Bicycle Facility.** An improvement or provision made by a public agency to accommodate or encourage bicycling, including a shared roadway not specifically designated for bicycle use, or a parking and storage facility.

4. **Bicycle Lane.** A portion of a roadway which has been designated by means of signing, striping, or other pavement markings for the preferential or exclusive use of bicyclists. It is distinguished from the motor-vehicle travel portion of the roadway by means of a physical barrier or pavement marking. A bicycle lane can also assume varying forms but it can be either of the following:

   a. bicycle lane between parking lane and travel lane; or
   b. bicycle lane between roadway edge and travel lane, where parking is prohibited.

5. **Bikeway.** A road, path, or way which is specifically designed for bicycle travel, regardless of whether such facility is designated for the exclusive use of bicyclists or is shared with other non-motorized transportation modes.

6. **Rail-Trail.** A shared-use path, either paved or unpaved, built within the right of way of an abandoned railroad.

7. **Trail.** This term can have different meanings depending on the context, but it does not have the same meaning as the term *shared-use path.* A trail can be used for exercise, transportation, recreation, or education. A trail user can be a hiker, bicyclist, skater, equestrian, snowmobiler, pedestrian, or other user. A trail that is designed to provide a bicycle-transportation function while supporting multiple users is called a shared-use path. Where a trail is designated as a bicycle facility, the design criteria for a shared-use-path should be satisfied.
8. **Greenway.** This is a linear space established along a corridor, such as a riverfront, stream valley, or other natural or landscaped system. A greenway can connect open spaces, parks, nature reserves, cultural features, historic sites with populated areas, or with one another. A greenway can or cannot include a bikeway or shared-use path.

9. **Roadway.** The portion of a highway, including shoulders, intended for vehicular use.

10. **Shared Roadway.** A roadway which is open to both bicycle and motor-vehicle travel. It can be an existing roadway or street with a wide curb lane or a road with paved shoulders.

11. **Shared-Use Path.** A facility that is physically separated from motorized-vehicular traffic with an open space or barrier and either within the highway right of way or within an independent right of way. A shared-use path can also be used by pedestrians, skaters, wheelchair users, joggers, or other non-motorized users. A shared-use path may assume a different form as conditions warrant. It may be a 2-direction, multilane facility, or where it parallels a roadway with limited right of way, a single lane on each side of the roadway.

12. **Sidewalk.** The portion of a street or highway right of way designed for preferential or exclusive use by pedestrians.

13. **Signed Shared Roadway.** A shared roadway which has been designated by means of signing as a preferred route for bicycle use.

### 51-7.02 Local Public Agency Coordination

A local public agency will determine the type of bicycle facility and its location during the planning stages. If it is determined that a bicycle facility is feasible and can be properly funded, the designer should coordinate with the agency in the design of the facility.

### 51-7.03 General Design Factors to be Considered

The Department’s goals include encouraging and accommodating safe bicycling. From a design perspective, these goals are achieved by first having an understanding of the dimensions of a bicycle and bicyclist and the operational characteristics. These design considerations are critical in planning and designing an on-road or off-road shared-use facility.
51-7.03(01) Bicycle Operating Space and Characteristics

Users other than bicyclists must be considered in the design of a shared-use path or greenway. Such other users include inline skaters, adult tricyclists, bicyclists with trailers, recumbent bicyclists, pedestrians (including runners, walkers, joggers, etc.), or wheelchair users. Other user types that are allowed to share the same space as a bicyclist should be integrated into the initial planning stages and the design and selection of the type of shared-use facility. See Figure 51-7A, User-Type Dimensions and Speeds.

To ensure the safety of the bicyclist and promote efficient bicycling, the dimensions of the bicycle and bicyclist must be considered, along with the amount of lateral and vertical clearance needed, in the planning and design of a shared-use facility. The bicycle and bicyclist dimensions and the lateral and vertical clearance have a direct bearing on the amount of right of way required to accommodate bicycle traffic.

The dimensions of a typical bicycle are a handlebar height of 2.5 to 3.5 ft, handlebar width of 2 ft, and bicycle length of 5 to 6 ft. A tandem or recumbent bicycle can have different dimensions. A typical bicycle with an attached trailer is 3.7 to 4.3 ft wide and 8.5 to 9.5 ft long.

A moving bicyclist requires a horizontal corridor at least 3 ft width to maintain balance if riding at a low speed or against a crosswind. To ride comfortably and to avoid a fixed object such as a sidewalk, shrub, pothole, sign, signal, etc., or another user such as a pedestrian or in-line skater, a bicyclist requires at least an additional 1 ft of lateral clearance on each side, to make the total operating width of a one-way corridor as 5 ft.

If space is restricted, such as in a tunnel or bridge, a width of at least 10 ft is recommended for two opposing bicyclists to comfortably pass each other. See Figure 51-7B, Bicyclist Operating Space. More width may be required to accommodate an in-line skater, bicycle with trailer, etc. Space is necessary for a bicyclist to react to an unexpected maneuver of another bicyclist or other user. Other users and their dimensions and operational characteristics should be considered in addition to bicyclists in designing the facility. Pedestrians often walk or jog on paths or trails in pairs, side by side.

A bicyclist can maintain a cruising speed of 12 to 25 mph and can maintain a speed of 20 mph or higher on flat terrain and with windless conditions. In a descent with a tailwind, a bicyclist can reach a speed in excess of 30 mph.

A shared-use facility should be designed with the gentlest slopes practical to encourage its use. However, facility design and the bicyclist’s behavior can be adjusted to compensate for steep terrain. Elevation changes can appeal to some bicyclists.
Stopping distance and lack of traction can influence the design of a curve on a shared-use facility.

A motorist may not see, or expect to see, a bicyclist, especially after dark or in inclement weather. Each intersection or roadside requires adequate sightlines and lighting to help increase the motorist’s visibility of the bicyclist.

51-7.03(02) Types of Bicyclists

Bicyclists’ skills, confidence, and preferences vary significantly. A shared-use facility should be planned to provide continuity and consistency for all types of bicyclists.

1. **Advanced Bicyclist.** An advanced bicyclist is experienced, and uses his or her bicycle as he or she does a motor vehicle. He or she is cycling for convenience and speed and wants direct access to the destination with minimum detour or delay. He or she rides with motor-vehicle traffic, on the roadway, but needs sufficient operating space to eliminate potential conflicts with passing motor vehicles.

2. **Basic Bicyclist.** A basic bicyclist is an adult or teenager, new or casual to cycling, who is less willing or able to ride in motor-vehicle traffic without a feature such as a bike lane, paved shoulder, or road with lower motorized-vehicle speeds or traffic volume. He or she prefers to avoid a road with higher motor-vehicle speeds or traffic volume unless there is ample roadway width to allow motor vehicles to pass. He or she prefers direct access to the destination using a road with low motor-vehicle speeds or traffic volume, a bike lane, a wide paved shoulder, or a shared-use path.

3. **Child Bicyclist.** A child is a teenager or younger, who rides with or without adult supervision. A child’s cycling can be initially monitored by an adult. The child is eventually allowed independent access to the road system. A child still requires access to each destination relative to a residential area, including school, recreational facility, or shopping area. A residential street with low motor-vehicle speeds, linked with shared-use paths and other streets with pavement markings between bicycles and motor vehicles, can accommodate a child bicyclist. A child needs supervision, a basic knowledge of traffic laws, and better than basic bicycle-operating skills before he or she can safely use an on-road bicycle lane with high motor-vehicle speeds or traffic volume.

The selection of the facility type suited for a travel corridor depends on bicyclists’ abilities, corridor conditions, current and future land use, topography, population growth, roadway characteristics, and the cost to build and maintain the facility. Within a travel corridor, more than one option may
be needed to serve all bicyclists or other users as appropriate. However, no single type of shared-use facility or road design suits every bicyclist.

For a basic or child bicyclist as described above, key travel corridors should be identified through a planning process, and bicycle accommodation should be provided through such corridors. However, roads and shared-use paths that are not on the bicycle-network plan that link residential areas to schools, libraries, shopping areas, employment centers, or parks are also critical in serving the basic or child bicyclist. Development of a facility that includes wide curb lanes or paved shoulders to accommodate bicyclists will help build the continuity of the bicycle network.

51-7.03(03) Share-Use-Path Type Selection

Guidelines are provided herein for the design of a shared-use path that accommodates the operating characteristics of a bicyclist as described in Section 51-7.03(01). Modifications to a facility (e.g., width, curve radius, superelevation, etc.) that are necessary to accommodate an adult tricyclist, bicyclist with trailer, tandem bicyclist, recumbent bicyclist, or other special-purpose human-powered vehicle user and accessories should be made in accordance with the expected use, using engineering judgment.

51-7.03(04) Accessibility Design

A transportation facility such as a path, sidewalk, or bicycle facility shared with pedestrians is required to comply with the Americans with Disabilities Act of 1990 (ADA) so that it is functional for all users, both with and without disabilities. ADA is a law that protects the civil rights of persons with disabilities. New construction with a bicycle or pedestrian facility must incorporate accessible pedestrian features in accordance with ADA.

To optimize design for a person with a disability, surface cross slope, surface material treatment, minimum path width, maximum grade, curb-ramp locations and design, or other elements must be addressed that can create localized obstructions affecting the facility’s use. Removal of all accessibility barriers will maximize opportunities for the largest number of people.

The Access Board, the federal body responsible for drafting accessibility guidelines, is working to supplement such guidelines that it has issued for the built environment and will address unique constraints specific to public rights of way. Once finalized, such guidelines will become a part of the ADA Accessibility Guidelines for Buildings and Facilities (ADAAG). The guidelines being developed include surface treatment, minimum path width, passing space, and changes in the level surface.
51-7.04 Types of Bicycle Facilities

Each type of facility has advantages and disadvantages. The following guidelines are provided to assist in making decisions regarding bicycle-facility type. The use of definite, numerical limits for warrants should be avoided. Placing emphasis on a single concern should be avoided. Each route is unique and must be evaluated individually.

This Section is organized based on the classifications of bikeways as follows:

1. bikeway;
2. bicycle lane;
3. shared roadway;
4. signed shared roadway;
5. trail or greenway;
6. shared-use path; or
7. path for other use.

Where guidelines overlap across classifications, reference is made to the appropriate Section herein.

51-7.04(01) Bikeway

A bikeway is constructed explicitly for the use of bicyclists. The cyclist is provided with a clear-cut route and is protected from hazardous conflicts. However, this type of facility is expensive to construct due to right-of-way and construction costs compared to a bicycle lane, shared roadway, or signed shared roadway. A bicycle lane, shared roadway, or signed shared roadway does not typically require the acquisition of the extra right of way.

Justifications for a bikeway include the following:

1. vehicular speed of 50 mph or higher on adjacent roadway;
2. AADT of higher than 2000 on adjacent roadway;
3. trucks 10% DHV or higher than on the adjacent roadway;

4. high bicycle-traffic volume;

5. substantial anticipated increase in vehicular- or bicycle-traffic volume;

6. absence of suitable alternate routes;

7. demonstration that the facility can serve a definite purpose;

8. a large number of curb cuts on the adjacent roadway vs. a low number of curb cuts or intersecting streets on the bikeway; or

9. reasonable indication that a bicycle path is the safest and most economical method of providing a bicycle facility.

Items 1, 2, and 3 above are subject to the accepted roadway values for high speed, high traffic volume, or high percent trucks for a specific roadway type or locations.

51-7.04(02) Bicycle Lane

The occupation of a portion of a roadway by a bicycle lane implies a reasonable degree of safety for the cyclist. Conditions must be less severe than those which recommend a bicycle path. The use of a bicycle lane is normally restricted to bicycles, but exceptions may be made. A physical or symbolic barrier must be used to delineate the bicycle portion of the roadway. This is ordinarily a non-skid painted stripe on the roadway surface.

An advantage of a bicycle lane is the relatively minor right-of-way requirement. It can be provided where the construction of a bicycle path is impractical. A bicycle lane, although not ideal, may be the most practical means of developing a bikeway.

Justifications for a bicycle lane include the following:

1. vehicular speed of 45 mph or lower on the adjacent roadway;

2. AADT of 2000 or lower on the adjacent roadway;

3. moderate bicycle-traffic volume;
4. sufficient land to construct bicycle lane without major disruptions on the surroundings;

5. demonstration that the facility serves a definite purpose; or

6. indication that a bicycle lane is safe and feasible.

51-7.04(03) Shared Roadway

Having bicyclists and motorists share the same lane can be a practical method of establishing a bikeway if some of the justifications listed below can be satisfied. Because a shared roadway is designated only by means of Bike Route signs, it is implied that the roadway provides safe conditions for both cyclist and motorist. Where a bikeway is warranted, a shared roadway should be permitted only where the existing conditions either do not justify the greater expense of a higher type facility or prevent its installation.

Justifications for a shared roadway include the following:

1. vehicular speed of 45 mph or lower on the roadway;

2. AADT of 2000 or lower on the roadway;

3. trucks 10% DHV or less on the roadway;

4. moderate bicycle-traffic volume;

5. demonstration that the facility serves a definite purpose;

6. a higher-grade facility is not warranted for bicycles, or

7. indication that a shared roadway is a safe and feasible method of providing this type of bikeway.

51-7.04(04) Signed Shared Roadway

A signed shared roadway is designated with “Bike Route” signs. It serves either to provide continuity to other bicycle facilities (usually bike lanes), or to designate a preferred route through a high-demand corridor.
As with a bike lane, signing of a shared roadway can indicate to a bicyclist that particular advantages exist to using such a route compared with an alternative route. Signing also serves to advise vehicle drivers that bicyclists are present.

### 51-7.05 Shared-Use Path

Shared-use path is a term that has been incorporated into the AASHTO *Guide for the Development of Bicycle Facilities* in recognition that such a facility is used by multiple non-motorized users. A shared-use path is located on exclusive right of way, with no fixed objects in it and minimal cross flow by motor vehicles. Portions of a shared-use path may be within the road’s right of way but physically separated from the road by means of a barrier or landscaping. Users include bicyclists, in-line skaters, wheelchair users (both non-motorized and motorized), and pedestrians including walkers, runners, or people with baby strollers or dogs. In order to provide a sense of security and safety to pedestrians, bicyclists, or wheelchair users as well as other shared-use path users, equestrian use should not be permitted on a paved shared-use path as horses can behave unpredictably, and become uncontrollable. This type of incident can become a safety hazard to the users.

### 51-7.05(01) Shared-Use-Path Special Guidelines

The guidelines described below are intended to be applied using a flexible design approach. Design speed; path width; structural capacity for a new, rehabilitated, or existing bridge to remain in place; minimum vertical clearance; bridge-railing safety performance; and accessibility requirements for handicapped individuals are the only Level One design criteria. All other design features are considered to be Level Two design criteria. Where the Level One design criteria cannot be satisfied, the designer should submit a design exception request to the Roadway Services Office manager for review and approval. Where the Level Two design criteria cannot be satisfied, the designer should document in the project file that such design criteria have not been satisfied, and should provide a brief rationale for not satisfying the Level Two design criteria. An in-depth documentation to justify a design decision involving failure to satisfy minimum design standards is not necessary. The designer may review, for general information, Sections 40-8.0, which provides a discussion of Level One and Level Two design criteria and the design exception process for highways and streets where the design standards cannot be satisfied due to limited availability of right of way or other constraints.

A shared-use path is designed for two-way travel except under certain conditions. The guidance described herein is for a two-way facility unless otherwise stated.
51-7.05(02) Shared-Use-Path Design Considerations

1. **Separation Between Path and Roadway.** Where a two-way shared-use path is located adjacent to a roadway, a wide separation between the shared-use path and adjacent highway is desirable, demonstrating to both the path user and the motorist that it functions as an independent facility. The factors in determining how far away a shared-use path should be separated from the roadway include the posted speed limit of the road, the types of signs between the path and roadway, the amount of space available, and whether the roadway has a rural shoulder-and-ditch cross section or an urban curb-and-gutter cross section.

The separation distance between a path and a roadway depends primarily on the posted speed limit of the road. The recommended separation for a rural shoulder-and-ditch road cross section is identified in Figures 51-7C and 51-7D. The recommended separation for an urban curb-and-gutter road cross section is identified in Figures 51-7E and 51-7F. Where space is limited or a constraint exists and the recommended separation distances between a path and a roadway cannot be attained, the Level Two design-criteria requirements as described in Section 51-7.05(01) will apply.

A traffic barrier is desirable for path-user safety if the separation distance between the edges of the roadway and shared-use-path shoulders is less than indicated in Figure 51-7D or 51-7F. The appropriate type of traffic barrier will depend on motor-vehicle speed. Where a concrete traffic barrier is adjacent to a shared-use path, a minimum clearance, paved width of 1 ft, is recommended, or a desirable width of 3 ft can be provided. For guardrail supported on posts, a clearance of 3 ft or greater from the edge of the shared-use path’s pavement is recommended because of the greater risk of injury to a path user striking a post. The back sides of posts next to or parallel to a shared-use path should be provided with a rubrail to minimize the possibility of a path user snagging on a post. Bridge railing must satisfy the guidelines provided in Section 61-6.0 or as otherwise required. See Section 51-7.07 for additional information regarding bridge railing or barrier.

2. **Snow Storage in Separation Area.** Where snow storage is an issue, the designer should contact the district Office of Roadway Services. The separation area between a road and a shared-use path may be used to store snow that has been removed from both the roadway and the path. The separation-area width should be 18 ft. Where space is limited, the likely amount of removed snow, the space needed to store it, and how stored snow will be managed should be considered in the overall road cross-section design. If
snow is stored in the separation area, at least 75% of the path’s width should remain usable.

3. **Design Speed.** A bicycle minimum design speed of 15 mph is required. A bicycle design speed of 20 mph is desirable. For a descending grade of 500 ft or longer and 4% or steeper, a bicycle design speed of 30 mph is desirable. The selected design speed should be maintained throughout the length of the shared-use path. Alternating design speeds are not recommended. If site conditions will not allow the appropriate path geometrics for the selected design speed, a lower design speed should be selected for the path except where a portion of it is in a rural area and another portion is in an urban area.

4. **Width and Lateral Clearance.** The overall width of a shared-use path includes the pavement width, graded shoulders on both sides, and an additional clear width beyond the shoulders. Determining an appropriate pavement width requires project-specific evaluation, as discussed below.

   Width, pavement design, and clearances should accommodate a maintenance or emergency vehicle, such as a pickup truck, mower, ambulance, etc. The paved path should be wider than the widest anticipated vehicle to avoid pavement-edge deterioration.

   The desired paved width is 10 ft. The minimum paved width is 8 ft. It may be necessary or desirable to increase the width to 12 ft or 14 ft; see Figure 51-7G, Path Pavement Width Based on Path-User Travel Composition. A clear width of at least 3 ft is desirable beyond the edge of the paved portion to provide clearance from trees, poles, walls, fences, guardrails, or other lateral obstructions.

   A minimum clear graded shoulder width of 2 ft beyond the edge of the shared-use-path pavement with a maximum slope of 6:1 should be maintained adjacent to both sides of the path. Where the path is adjacent to a canal, ditch, or embankment downslope which is steeper than 3:1, a wider graded shoulder should be considered. A clear width of 5 ft minimum from the edge of the pavement to the shoulder break point is desirable. A physical barrier such as a railing, fence, or dense shrubbery may be required near the outer edge of the graded shoulder at the top of the embankment slope if obstacles are present on at the bottom of the embankment. Where such clear width is less than 5 ft, the embankment slopes and associated dropoffs where a physical barrier should be considered are as follows:

   a. the slope is 3:1 or steeper and the dropoff is at least 6 ft;

   b. the slope is 2:1 or steeper and the dropoff is at least 4 ft; or
c. the slope is 1:1 or steeper and the dropoff is at least 2 ft.

The minimum standard safety-railing height is 42 in. For parameters greater than those listed above, a 54-in. maximum safety-railing height should be considered. The railing should not present an injury hazard to the shared-use path user.

5. **Vertical Clearance.** The design vertical height for a bicyclist is 8 ft. Though a tall individual will not reach this height if seated on a bicycle, extra clearance must be allowed for a bicyclist pedaling upright or passing under an overpass. Vertical clearance should be a minimum of 10 ft to allow for the clearance of a maintenance or emergency vehicle in an underpass or tunnel, and to allow for overhead signing. A shared-use-path structure over a vehicular roadway should have a minimum vertical clearance of 17.35 ft plus 0.15 ft for future resurfacing of the vehicular roadway. For additional information on vertical clearance, see Figure 44-4A.

Where an existing bridge structure, such as that on an abandoned railroad corridor, is to be utilized to cross over a vehicular roadway, pedestrian walkway, or trail, the vertical clearance should satisfy the applicable criteria shown in Figure 44-4A, Sections 54-3.02(03), 54-5.02, 56-4.09(01), and 56-4.09(02), and Figures 56-4E and 56-4F, as applicable.

6. **Profile Grade.** The profile grade should be kept to a minimum, especially on a long incline. A grade of steeper than 5% is undesirable because an ascent is difficult for many bicyclists to climb and a descent causes some bicyclists to exceed the speed at which they are competent or comfortable. Where terrain dictates, the 5% grade may be exceeded for short lengths of the shared-use path.

The AASHTO *Guide for the Development of Bicycle Facilities* acknowledges that on a recreational route, a 5% grade may be exceeded for a short length. The methods for mitigation of a steep grade are as follows:

a. eliminate hazards to the path user near the end of a steep downgrade or ramp;

b. warn the path user by means of signage ahead of a steep downgrade hazard;

c. provide signage stating the recommended descent speed;

d. exceed the minimum stopping sight distance; or
e. provide a series of short switchbacks near the top of a descent to contain the speed of a descending bicyclist, or consider a portion of 10 to 20 ft length with a 1 to 2% grade at the point of direction change on the switchbacks to provide a resting area for the path user.

f. A grade steeper than 8.3% exceeds the ADA Accessibility Guidelines for a pedestrian facility, and should be avoided on a shared-use path unless significant physical constraints exist.

See Figure 51-7H, Grade Restriction for Paved Shared-Use Path.

7. Horizontal Curvature and Superelevation. Unlike an automobile, a bicycle must be leaned while cornering to prevent it from falling outward due to the generation of centrifugal force. The balance of centrifugal force due to cornering, and the bicycle’s downward force due to its weight, act through the bicycle and the cyclist’s combined center of mass and must intersect a line that connects the front- and rear-tire contact points.

If a bicyclist pedals through a sharp curve and leans too far, a pedal will strike the ground due to a sharp lean angle. Although pedal heights are different for different bicycles, a pedal will strike the ground once the lean angle reaches about 25 deg. However, a basic bicyclist does not want to lean too drastically, therefore 15 deg is considered the maximum desirable lean angle. The maximum 20-deg lean angle shown in Figure 51-7I is applicable only if the bicyclist’s speed is 30 mph or higher. For a bicyclist who sits straight and firmly in the seat, and whose body is aligned with the vertical axis of the bicycle while pedaling through a curve, the minimum curve radius can be determined from the equation as follows:

\[
R = \frac{0.067V^2}{\tan \theta}
\]

(Equation 51-7.1)

Where:
- \(R\) = minimum radius of curvature (ft)
- \(V\) = design speed (mph)
- \(\theta\) = lean angle from the vertical (deg)

For the desirable maximum lean angle of 15 deg, \(R = 0.25V^2\).

The cross slope should not exceed 2 to 3% to satisfy the ADA requirements. Therefore, the maximum superelevation rate is 3%. In transitioning a 3% superelevation, a
minimum 25-ft transition distance should be provided between the end and the beginning of consecutive horizontal curves or reverse horizontal curves.

The minimum curve radius for a paved shared-use path can be determined from Figure 51-7I, based on a desirable maximum lean angle of 15 deg.

The coefficient of friction depends on speed; surface type, roughness and condition; tire type and condition; and whether the surface is wet or dry. The coefficient of friction should be selected based upon the point at which centrifugal force causes the bicyclist to recognize a feeling of discomfort and instinctively act to avoid higher speed. The coefficient of friction can vary from 0.31 at 12 mph to 0.21 at 30 mph.

Where a curve radius shorter than that shown in Figure 51-7I must be used due to limited right of way, topographical features, or other considerations, a lower design speed should be used. Curve warning signs and supplemental pavement markings should be installed in accordance with the MUTCD.

The amount of lateral clearance required on the inside of a horizontal curve is a function of the design speed, the radius of curvature, and the grade. The centerline of the inside lane is used in measuring the length of the bicyclist’s field of vision. Lateral clearance should be calculated based on the sum of the stopping sight distances for bicyclists traveling in opposite directions around the curve. If this sight distance cannot be provided, the path should be widened or a continuous centerline should be placed between the lanes for the entire length of the curve plus 30 ft beyond the curve at each end. See Figure 51-7J, Lateral Clearance at Horizontal Curve.

Figure 51-7K indicates the minimum lateral clearance that should be used to avoid line-of-sight obstructions for a horizontal curve.

8. Vertical Curvature and Stopping Sight Distance. To provide a path user with an opportunity to see and react to the unexpected, a shared-use path should be designed with adequate stopping sight distances. The distance required to bring a bicycle to a full controlled stop is a function of the bicyclist’s perception and brake reaction time, the initial speed of the bicycle, the coefficient of friction between the tires and the pavement, temperature and moisture conditions, the braking ability of the bicycle, the grade, and the bicyclist’s weight and equipment.

Figure 51-7L indicates the minimum stopping sight distance for the appropriate design speed and profile grade based on a total perception and brake reaction time of 2.5 s and a coefficient of friction of 0.25 to account for the poor wet-weather braking characteristics.
of many bicycles. For a two-way shared-use path, the sight distance in the descending direction, where \( G \) is negative, will control the design.

Sight distance at a grade crest can be checked using Figure 51-7M, Minimum Crest Vertical Curve Minimum Length, \( L \), Based on Stopping Sight Distance, \( S \), or its associated equations.

A longer vertical curve should be provided where practical. The equations are based on an eye height of 4.5 ft and an object height of zero. An object as small as gravel on the surface can be hazardous to a bicyclist.

Figure 51-7N provides the stopping sight distance for a downgrade.

51-7.06 Pavement Section [Rev. Jan. 2011]

A hard, all-weather pavement surface is preferred to that of crushed aggregate, sand, clay or stabilized earth, since these materials provide a much lower level of service and require higher maintenance. In an area that is subjected to frequent or occasional flooding or drainage problems, or in an area of steep terrain, an unpaved surface will often erode, so therefore it is not recommended.

A quality all-weather pavement structure can be constructed of hot-mix asphalt or portland-cement concrete. It is not practical to provide specific or recommended typical pavement sections that are applicable statewide, due to variations in soils, loads, materials, construction practices, or varying costs of pavement materials.

Designing and selecting the pavement section for a shared-use path is similar to designing and selecting a highway pavement section. The pavement section for a shared-use path should be designed with consideration given to the quality of the subsoil and anticipated loads. The principal loads will be from maintenance or emergency vehicles. These vehicles should be restricted to axle loads of less than 4.5 tons, especially in the spring.

The subgrade and pavement-section recommendations are subject to the approval of the Office of Pavement Engineering. Such approval should be included in the project’s design documentation.

A smooth riding surface should be constructed and maintained on a shared-use path. For a portland-cement-concrete pavement, the transverse joints necessary to control cracking should be saw cut to provide a smooth ride. However, skid-resistance qualities should not be sacrificed for the sake of smoothness. A broom-finished or burlap-dragged concrete surface is preferred.
If a motor vehicle is driven on a shared-use path, its wheels will often be at or near the edges of the path. This can cause edge damage that, in turn, will reduce the effective operating width of the path. A pavement width of less than 10 ft is not recommended. If a facility width of less than 10 ft is necessary; only narrower-track-width motor vehicles should be permitted on it.

At an unpaved roadway or drive crossing with a shared-use path, the roadway or drive should be paved for a minimum of 10 ft on each side of the crossing to reduce the amount of gravel being scattered onto the path by motor vehicles. The pavement structure at the crossing should be adequate to sustain the expected loading at that location.

Standard nonmotorized-use-facility typical sections have been developed for HMA and PCCP. An HMA pavement section has also been developed for use on an abandoned railroad corridor. The selection of HMA or PCCP will be determined by the designer. It will require neither a presentation to the Pavement Type Selection Committee nor an individual pavement-design review by the Office of Pavement Engineering.

The typical sections appear on the INDOT Standard Drawings series 502-NVUF and 604-NVUF for PCCP and HMA pavements, respectively.

51-7.07 Drainage

The recommended minimum pavement cross slope of 2% adequately provides for drainage. Sloping in one direction instead of crowning is preferred, and simplifies the drainage and surface construction. A smooth surface is essential to prevent water ponding or ice formation.

Where a shared-use path is constructed on the side of a hill, a ditch of suitable dimensions should be placed on the uphill side to intercept the hillside drainage. Such ditch should be designed so that an undue obstacle cannot interfere with bicycle traffic. Where necessary, a catch basin with drains should be provided to carry the intercepted water under the path. Drainage grates or manhole covers should be located outside the path. Each drainage-structure grate should have openings sufficiently narrow and short to prevent bicycle or wheelchair tires from dropping into it regardless of the direction of travel. To assist in preventing erosion in the area adjacent to the shared-use path, the design should include considerations for preserving the natural ground cover. Seeding, mulching, or sodding of adjacent slopes, swales, or other erodible areas should be shown on the plans.

51-7.07(01) Culvert
The minimum diameter of a culvert which conveys flow under a shared-use path is 15 in. Each culvert should be designed to pass a minimum 2-year event. The 100-year event is the design storm for backwater. The INDOT backwater policy will apply.

51-7.07(02) Bridge Structure

A structure crossing a stream with a drainage area of at least 1 mi² is considered a bridge, as an IDNR Construction in a Floodway Permit will be required. A bridge should be designed to pass a minimum 10-year event through the structure. The 100-year event is the design storm for backwater. The INDOT backwater policy will apply.

A trail crossing a stream should be designed so as not to create additional backwater for a 100-year event. A hydraulics model which satisfies the INDOT requirements will be required for the existing and proposed conditions. The trail crossing should be designed to prevent erosion and scour.

51-7.08 Grade-Separation Structure

A grade-separation structure can be necessary to provide continuity for a shared-use path.

This can either be stand-alone or in conjunction with a vehicular bridge.

For a new structure, it is desirable to match the approaching path width plus a 2-ft clear width on each side of the path. The minimum path width is 8 ft. If the minimum cannot be provided, a design-exception request should be prepared. Compromise in desirable design criteria can be inevitable due to the number of variables involved in retrofitting a shared-use path onto an existing grade-separation structure. Therefore, the clear-structure width to be provided is best determined by the designer for each structure. The following should be considered in determining the structure width:

1. it provides a minimum horizontal shy distance from a railing or barrier;

2. it provides needed maneuvering space to avoid a conflict with a pedestrian or other bicyclist who is stopped on the structure; and

3. it provides access by an emergency, patrol, or maintenance vehicle.
Vertical clearance is determined based on a motor vehicle’s use of the path. Where practical, a vertical clearance of 10 ft is desirable for adequate shy distance.

A railing, fence, or barrier on each side of a structure should be of at least 42 in. height. Increasing the barrier height to a maximum of 54 in. should be considered where conditions warrant.

Compromise in desirable design criteria can be inevitable due to the number of variables involved in retrofitting a shared-use path onto an existing grade-separation structure. Therefore, the clear-structure width to be provided is best determined by the designer for each structure, after considering the following.

1. Where a shared-use path is to be carried on a structure which is also used by motorists, and the motor-vehicle speed limit is at least 45 mph, a traffic barrier is required between the shared-use path and the motor-vehicle travel lanes, with a bicycle/pedestrian railing or combination railing on the outside edge of the structure. The type of required traffic barrier will depend on the speed of vehicular traffic. Additional considerations in selecting a barrier type include aesthetics, vehicular-traffic volume, or and the expected bicycle and pedestrian traffic volume.

2. Where a shared-use path on a raised sidewalk, or in a lane striped on the roadway next to a raised sidewalk, is to be carried on a structure which is also used by motorists, and the motor-vehicle speed limit is 40 mph or lower, a combination railing may be used on the outside edge of the structure without a traffic barrier between the roadway and the shared-use path. The sidewalk curb height should be 8 in. If there is no sidewalk, and the shared-use path is at the same elevation as the roadway, a traffic barrier or combination railing should be used between the roadway and the path, with a bicycle/pedestrian railing or combination railing at the outside edge of the structure.


A path-roadway intersection is among the most critical issues in shared-use-path design. According to the National Highway Traffic Safety Administration, more than half of all bicycle crashes occur at such intersections.

At an intersection, a bicyclist on a path faces many of the same conflicts as on a roadway, complicated by integration with pedestrians. Problems associated with an at-grade crossing often relate to the motorist’s expectation that crosswalk users will be traveling at pedestrian speeds rather than bicycle speeds.
For a motorist entering a path-roadway intersection, the motor-vehicle stopping sight distance requirements described in Section 46-10.0 must be satisfied.

A path intersection with a roadway offers many risks. If approaching a free right turn, a motorist does not anticipate a conflict on the right, and is looking to the left for traffic entering the intersection, so he or she may not see a bicyclist approaching the intersection on a parallel path. A turning motorist may not consider that a bicyclist will be traveling off the road, yet will be within the right of way. In encountering a motorist, a bicyclist is often compelled to stop and yield to a left- or right-turning vehicle. To account for this, an appropriate balance is found by locating the crossing close enough to the intersection to allow adequate motorist visibility, yet far enough away to allow sufficient motorist reaction time, but not so far away that an approaching motorist is unaware of the crossing bikeway. A one-way path at a signalized intersection can increase visibility and safety, especially regarding a right-turning motorist and a through-traveling bicyclist. The site specific elements that should be considered when making a decision are pedestrian volumes and types, traffic volumes, existing traffic control, posted speed limit, and geometric characteristics, e.g. the number of lanes, width of crossing, and visibility.

Figure 51-7.O indicates the treatment for a path-roadway intersection. Figure 51-7.O lists guidelines, not absolute requirements, for intersection treatment. Each intersection is unique and will require engineering judgment as to the appropriate solution.

The following should be considered in using Figure 51-7.O to select an intersection treatment.

7. The type of crossing used for bicycle or pedestrian traffic at an intersection between a main road and a secondary road is usually the same as for the main road.

8. If the number of lanes to be crossed is greater than 3 in each direction, or the total intersection width is greater than 75 ft, the intersection should have a pedestrian refuge or median island. Where a path user must wait on an island, a push button or bicycle-sensitive traffic detection device should be considered.

9. If the speed limit for a section of road without traffic signals is 45 mph or higher and it is not practical to provide a grade separation, reduction of the speed limit to 40 mph before the crossing, along with proper signing and lighting, should be considered.

10. In determining the need for, and suitability of, a grade separated crossing, the following criteria should be satisfied.
a. High vehicular volumes conflict with night pedestrian volumes, constituting an extreme hazard.

b. Modification of school routes, busing policies, campus procedures, or attendance boundaries to eliminate the need for a crossing is not feasible.

c. Physical conditions make a grade separation feasible from an engineering standpoint, including pedestrian channelization to ensure usage of the structure. In determining the location for a grade separation, the ramp grades on the path should be minimized, and the location should fit in with the rest of the path network.

d. Pedestrian crossings can be restricted for at least 600 ft on each side of the proposed pedestrian overpass.

e. A demonstrated problem exists that simpler, more economic solutions have failed to correct.

f. The anticipated benefits to be derived from the pedestrian overpass clearly outweigh the costs.

51-7.09(01) Intersection Types

Each intersection type may cross a number of roadway lanes, divided or undivided, with varying motor-vehicle speed and traffic volume, and may be uncontrolled or sign- or signal-controlled.

The types of path-roadway intersections are described below.

1. **Midblock Crossing.** Figure 51-7P shows an example of a midblock crossing. A midblock crossing should be far enough away from existing roadway intersections to be separated from an activity that can occur as a motorist approaches such an intersection, such as a merging movement, acceleration/deceleration, or preparation to enter a turn lane. Other considerations include right-of-way needs, traffic-control devices, sight distance for the path user and motorist, refuge-island use, access control, and pavement markings. These considerations are discussed below.

2. **Skewed Crossing.** A skewed crossing can be realigned to eliminate most or all of the skew. Figure 51-7Q shows a path realignment to achieve a 90-deg crossing. A maximum crossing angle of 45 deg is acceptable to minimize right-of-way requirements.
3. **Adjacent-Path Crossing.** An adjacent-path crossing occurs where a path crosses a roadway near an existing intersection of two roadways, whether it is a T-intersection including driveways, or a four-legged intersection, as shown in Figure 51-7R. This type of crossing should be located close to the roadway intersection so as to allow the motorist and path user to be able to recognize each other as intersecting traffic. With this configuration, the path user is faced with potential conflicts with a motorist turning left (movement A) or right (movement B) from the parallel roadway, and across or onto the crossing roadway (movements C, D, and E).

The major road may be either the parallel or crossing roadway. Right-of-way assignment, traffic control devices, and separation distance between the roadway and the path can affect the design of this intersection type. A further complication is the possibility of a conflict being unexpected by both the path user and the motorist. Sight lines across corners should be unobstructed.

For turning movement type A as shown in Figure 51-7R, left turns should be prohibited on a high-volume parallel roadway at a high-use-path crossing. For turning movement type B, the smallest practical turning radius may be required to reduce the motor-vehicle speed. A right turn on red should be prohibited for turning movement type B or D, with a stop line in advance of the path crossing.

4. **Complex Crossing.** A complex crossing consists of a configuration in which the path crosses directly through an existing intersection of two or more roadways, and there may be a number of motor-vehicle turning movements.

The treatments which may be considered include the following:

a. move the crossing;

b. install a traffic signal;

c. change the signalization timing; or

d. provide a refuge island to effect a two-step crossing for the path user.

Each situation should be treated as a unique challenge which requires creativity as well as engineering judgment. The safe passage of all modes of traffic through the intersection is the goal to be achieved.
5. **At-Grade Railroad Crossing.** Where a shared-use path crosses a railroad track, the safety of the path user should be ensured. The path should be straight and at a right angle to the rails. The more the crossing deviates from 90 deg, the greater the potential for a bicyclist’s front wheel to be trapped in the flangeway (the open space next to the rail) causing loss of control. If it is not practical for the crossing to be at 60 to 90 deg, the shared-use path should be widened to allow the user to cross as close to 90 deg as practical. See Figure 51-7S, Safe Railroad Crossing for Narrow-Wheeled Vehicle.

Narrow-wheeled vehicles, such as bicycles, wheelchairs, skateboards, etc., crossing rails at an angle of 30 deg or less are considered hazardous. The surface between the rails should be based on the planned uses of the roadway. A hot-mix asphalt or rubber surface is acceptable for an at-grade crossing.

The flangeways can be a safety concern and should be minimized. The field flangeway, or gap on the outside of a rail, can be reduced. A filler material of rubber or polymer can be installed to nearly eliminate the field flangeway and provide a level surface. The gauge flangeway, or the gap on the inside of the rail where the train wheel’s flange must travel, must be kept open. The minimum gauge-flangeway width for a public crossing is 2½ in., per American Railway Engineering and Maintenance Association regulations.

**51-7.09(02) General Guidelines for Intersection of Shared-Use Path with Road**

The following should be considered.

1. The shared-use path should intersect the road at a 90-deg angle.

2. The path width should be increased at the intersection approach to reduce user conflicts.

3. Clear sight lines should be provided for both the motorist and the path user.

4. Signage should be provided to alert the motorist of the path crossing.

5. A visible crosswalk should be provided across the roadway to increase path-user and motorist awareness.

6. Signs, both on the road and the path, should indicate whether the motorist or the path user has the right of way.
7. Curb ramps with detectable-warning devices are required to alert a path user with vision impairments of the street crossing.

8. An overpass, underpass, or facility on a highway bridge requires engineering feasibility and cost analysis to determine the most economical and effective means to provide continuity for a share-use path.

51-7.09(03) Other Intersection-Design Issues

Considerations to be made without regard to the type of path-roadway intersection described in Section 51-7.06 are as follows.

1. **Approach Treatment.** The shared-use path approaches to a roadway intersection should be on relatively flat grades. Stopping sight distance at an intersection should be evaluated. Adequate warning signs should be provided to allow a path user to stop before reaching the intersection, especially on a downgrade.

2. **Curb Ramps.** Sidewalk-type curb ramps should be the same width as the path. Curb cuts and ramps should provide a smooth transition between the path and the roadway.

   A 5-ft minimum or 10-ft desirable radius or flare should be considered to facilitate a right turn for a bicyclist. This consideration should also be applied to an intersection of two shared-use paths.

3. **Traffic-Control Devices.** A regulatory traffic-control device should be installed at each path-roadway intersection. The warrants described in the MUTCD, combined with engineering judgment should be considered in determining the type of traffic-control device to be installed.

   a. **Traffic Signal.** A traffic signal may be warranted. The MUTCD lists warrants for a traffic signal. It does not address a bikeway-roadway crossing. However, path traffic should be functionally classified as vehicular traffic, and each warrant should be applied accordingly.

      For a manually-operated signal-activation mechanism, the path-user signal button should be located where is easily accessible from the path and 4 ft above the ground so that a bicyclist need not dismount to activate the signal. For a signalized divided-roadway intersection, a push button should also be located in the median to account for a path user who is trapped in the refuge area.
2013 Indiana Design Manual, Ch. 51

b. “Stop” Sign. A “Stop” sign should be placed as close to the intended stopping point as possible and should be supplemented with a stop line. A four-way stop is not recommended due to frequent confusion about, or disregard for, path-user or motorist right-of-way rules. Sign type, size, and location should be in accordance with the MUTCD. A shared-use-path “Stop” sign should be located such that a motorist is not led to stop at such a sign. A roadway “Stop” sign should be located such that a path user is not led to stop at such a sign.

4. Transition Zone at Path Termination. Where a shared-use path terminates at an existing road, it should be integrated into the existing system of roadways. The path terminal should be designed to transition path traffic into a safe merging or diverging situation. Appropriate signage is necessary to warn and direct both the path user and the motorist regarding the transition zone.

5. Refuge Island. A refuge island should be considered for a path-roadway intersection in which one or more conditions apply as follows:

   a. a high roadway traffic volume or speed creates unacceptable conditions for the path user;

   b. the roadway is wider than 75 ft, or a pedestrian walking at 2.5 ft/s cannot completely cross the street during the green traffic-signal phase;

   c. a mid-block shared-use-path crossing or a path-roadway intersection is located where there are limited gaps in traffic; or

   d. the crossing will be used by a number of people who cross relatively slowly, such as the elderly, schoolchildren, persons with disabilities, etc.

The refuge area should be large enough to accommodate platoons of users, including groups of pedestrians, groups of bicyclists, individual tandem bicycles which are longer than standard bicycles, wheelchair users, or people with baby strollers. The area may be designed with the storage aligned across the island or longitudinally. See the example in Figure 51-7T, Refuge Island at Roadway Intersection. Adequate space should be provided so that those in the refuge area do not feel threatened by passing motor vehicles while waiting to finish crossing the roadway.

A refuge island allows a path user to cross one direction of driving lanes, then rest and assess when he or she is able to complete the roadway crossing. A refuge island provides a sense of
security to a pedestrian crossing a busy roadway with few gaps in traffic. A refuge island is typically used at a mid-block crossing, but is also acceptable to use at a path-roadway intersection.

A raised island should be cut through level with the roadway, or have curb ramps at both sides to comply with ADA, and a level area of at least 4 ft width between the curb ramps. A refuge island should be of at least 8 ft width where used by a path user. There should be at least 6.5 ft on each side of the cut-through. A path user should have a clear line of visibility on the island and should not be obstructed nor restricted by poles, sign posts, utility boxes, etc. The desirable width of the island and the width of the crosswalk equal to the shared-use-path width at the island are illustrated in Figure 51-7T.

51-7.09(04) Restriction of Motor-Vehicle Traffic

A shared-use path requires a physical barrier at a highway intersection to prevent an unauthorized motor vehicle from using the facility. A lockable, removable, or reclining barrier post can be used to permit entrance by an authorized motor vehicle. Each post or bollard should be set back outside the intersecting-highway clear zone or be of a breakaway design. The post should be reflectorized for nighttime visibility and painted a bright color for improved daytime visibility. Striping an envelope around the post is recommended as shown in Figure 51-7U. Where more than one post is used, an odd number of posts at 5 ft spacing is desirable. A wider spacing can allow entry by a motor vehicle, but a narrower spacing can prevent entry by an adult tricyclist, wheelchair user, tandem bicyclist, recumbent bicyclist, or bicyclist with a trailer.

Another method of restricting motor-vehicle entry is to split the entryway into 5-ft sections separated by low landscaping. An authorized motor vehicle can still enter if necessary by straddling the landscaping. The higher maintenance costs associated with landscaping should be considered before this method is selected.

51-7.10 Signing and Marking

Adequate signing and marking are essential on a shared-use path, especially to alert the user to potential conflicts and to convey regulatory messages to both the path user and motorist at a highway intersection. Guide signing to indicate direction, destination, distance, or route number or name of intersecting street, should be used in the same manner as on a highway. A uniform application of traffic-control devices, as described in the MUTCD, provides minimum traffic-control measures which should be applied.
A yellow center line of 4 in. width should be considered to separate opposite directions of travel. The stripe should be broken where adequate passing sight distance exists, and solid elsewhere, or where passing by bicycles is to be discouraged, as follows:

1. for high traffic volume of bicyclists or other path users;
2. on a horizontal or vertical curve with restricted sight distance; or
3. on an unlighted path where nighttime riding is expected.

White edge lines are beneficial where bicycle traffic is expected during early-morning or early-evening hours.

Further guidance on signing and marking is provided in the MUTCD.

51-7.11 Lighting

Lighting should be considered where night usage is expected, including an area serving college students or commuters, or at a highway intersection. Fixed-source lighting reduces crashes along a shared-use path or at an intersection with a roadway, and allows the user to see the path’s direction, surface condition, or obstacles.

Lighting a shared-use path permits some freedom in system and luminaire design. Lighting should be provided that fits the site’s needs and satisfies the recommendations described below.

The lighting system as a whole should provide adequate horizontal and vertical illumination along the entire length and width of the facility without significant variations in luminous intensity to which a path user or motorist can experience difficulty adjusting to. Horizontal illumination, measured at pavement level, enables a path user to understand pavement markings and to be able to easily follow the path. Vertical lighting, with illumination level measured 6 ft above the pavement, is most effective for illuminating the path and obstacles.

To avoid sharp differences in brightness, the uniformity ratio of illumination is determined by dividing the average illumination level by the minimum illumination level.

At a roadway intersection, illuminating the shared-use path for 75 ft on either side of the roadway is desirable. Transitional lighting is recommended on an unlit roadway crossed by a shared-use path.
Figure 51-7V indicates the average maintained luminance level and should be considered a minimum, particularly if security or the ability to identify path users from a distance is important. Figure 51-7V should be used for a shared-use path that is straight and level or has only minor curves or grade changes. Additional illumination is required where visibility is limited or where complex maneuvering can occur (i.e., abrupt curve, grade, roadway intersection, interchange, overpass, or underpass). A shared-use path which crosses a roadway in the middle of a long block or at an intersection of two roadways should receive additional illumination. Lighting should be provided at each warning-sign location where electricity is accessible. A light pole must satisfy the recommended horizontal and vertical clearances as outlined for other obstructions along the path. Lighting fixtures should be to a scale appropriate for a shared-use path.

51-7.12 Bicycle-Parking Facility

Providing a bicycle-parking facility is an essential element in an overall effort to promote bicycling. Some people are discouraged from bicycling unless adequate parking is available. A bicycle-parking facility should be provided at both the trip origin and trip destination and should offer protection from theft and damage. Bicycle parking can be long-term or short-term. The minimum requirements for each differ in their placement and protection.

A short-term facility is for decentralized parking where the bicycle is left for a short period of time and is visible and convenient to a building entrance.

A bicycle rack should be provided if it satisfies the following:

1. does not bend wheels or damage other bicycle parts;
2. accommodates a high-security U-shaped bicycle lock;
3. is visible to passers-by to promote usage and enhance security; and
4. has as few moving parts as possible.

A parking facility should be able to accommodate a wide range of bicycle shapes and sizes, including tricycles or trailers if used locally. A facility should be easy to operate. If possible, signs depicting how to operate the facility should be posted.
51-8.0 LANDSCAPING

51-8.01 General

Roadside landscaping can greatly enhance the aesthetic value of a highway. Landscaping treatments should be considered early in project development so that they can be easily and inexpensively incorporated into the project design. This may require the acquisition of additional right of way to implement these treatments.

Landscaping treatments are typically not included with other project types, but are generally completed as a separate project. Landscaping treatments will be considered on a project-by-project assessment.

51-8.01(01) Responsibility

The Production Management Division’s Services and Cultural Resources Team has the primary responsibility for determining or reviewing landscaping treatment. During the final field check, a landscape architect will attend to determine the landscaping treatment. The Services and Cultural Resources Team or landscape consultant will submit recommendations and landscaping details to the designer for incorporation into the project design.

51-8.01(02) References

For information on landscaping procedures and plants, the designer should contact the Services and Cultural Resources Team for their expertise. The designer should review the INDOT Standard Drawings series 622-LSPL and 622-LSPR, the AASHTO A Guide for Transportation Landscape and Environmental Design, and the Team’s reference library for more information on landscaping.

51-8.02 Benefits

Roadside landscaping can be designed advantageously to yield several benefits. The most important objective is to naturally fit the highway into the existing terrain. The existing landscape should be retained to the maximum extent practical. The following is a brief discussion of the benefits of proper landscaping.

1. **Aesthetics.** Gentle slopes, hills, parks, bodies of water and vegetation have an obvious aesthetic appeal to the highway user. Landscaping techniques can be used effectively to
enhance the view from the highway. In a rural area, the landscaping should be natural and should eliminate construction scars. The planting shape and spacing should be irregular to avoid a cosmetic appearance.

In an urban area, the smaller details of the landscape predominate and plantings become more formal. The interaction between the occupants of slow-moving vehicles and pedestrians with the landscape determines the scale of the aesthetic details. The designer may be able to provide walking areas, small parks, etc. Landscaping should be pleasant, neat, and sometimes ornamental, and it should require low maintenance.

2. **Erosion.** Landscaping and erosion control are strongly interrelated. Flat and rounded slopes and vegetation serve to both prevent erosion and provide aesthetic value. Chapter 37 provides additional information on erosion control.

3. **Maintenance.** Landscaping decisions will greatly affect roadside maintenance. Maintenance activities for mowing, fertilizing, or using herbicides should be considered when designing the roadside landscape. Involvement by other public or private groups (except on an Interstate route) should be encouraged to enhance the roadside landscape (e.g., Adopt-A-Highway Program).

4. **Screening for Headlight Glare.** Depending upon roadway alignment and the selected type of vegetation, landscaping features may be used to effectively screen headlight glare, for example, in a freeway median.

5. **Screening for Noise Abatement.** Although the effect may be more psychological than real, landscaping features may have some masking benefits to sensitive receptors.

6. **Screening of Undesirable View.** Screening of a junkyard or other undesirable view may be enhanced through the use of landscaping features.

7. **Snow Drift.** Landscaping features may assist in preventing snow from drifting and accumulating on the roadway.

**51-8.03 Landscaping Considerations**

All landscaping activities should be properly coordinated with other project design elements. The objectives are that other design elements should not be compromised by landscaping, and secondary benefits may be gained by the proper application of the landscaping features. Examples of coordination between landscaping and project design are briefly discussed below.
1. **Geometric Design.** On a new-construction or reconstruction project, the geometric design of the highway should be blended to fit the natural topography and landscaping features of the area. As practical, existing landscaping elements should be preserved and enhanced. The roadway alignment and cross-section design should be compatible with the landscaping objectives. The landscaping treatment should not be made to interfere with the driver’s horizontal and intersection sight distances.

2. **Roadside Safety.** The introduction of landscaping features should not compromise the objectives of roadside safety. Chapter 49 provides the Department’s criteria for roadside-safety design. The most significant roadside-safety element relative to the use of landscaping features is the clear-zone concept. Roadside hazards should not be located within the designated clear zone. A tree is considered a roadside hazard.

3. **Environmental.** Every effort should be made to use vegetation that will survive in the area with minimum maintenance. The selection of the vegetation will depend upon the soil conditions, drainage, amount of sun exposure, diseases and insects, road deicing chemicals, temperature, and pollution.

4. **Economics.** Plant selection, availability, quantity, and size greatly affect the cost of landscaping. The selection of the plantings should be so as to provide a cost-effective design.

**51-8.04 INDOT Landscaping Policy**

**51-8.04(01) Plant-Establishment Policy**

A project which includes plantings may include a special provision which requires the contractor to be responsible for a plant-establishment period of at least one year. A longer establishment period may be required where survival is considered essential to the function of the plantings (e.g., junkyard screening, urban landscaping).

**51-8.04(02) Protection of Existing Vegetation**

Wherever practical, existing trees or other landscaping features should not be removed. This objective, however, must be compatible with other considerations such as roadside safety, geometric design, utilities, terrain, public acceptance, and costs. The plans should clearly designate all existing landscape features which will be retained. If the existing plant material conflicts with
these considerations, where applicable, the plant material should be evaluated by a landscape architect for possible relocation to a more suitable portion of the right of way.

51-8.04(03) Disturbed Area

In an area disturbed by construction work, the designer should specify that the turf be reestablished. Turf establishment refers to the revegetation of a disturbed area. The designer should use the following guidance to determine the appropriate turf establishment, depending upon individual site conditions.

1. **Topsoil.** Topsoil is placed in a disturbed area to a depth of 6 in. or greater depending upon the underlying soil conditions.

2. **Planting of Grass.** Each area disturbed by construction, except exposed rock surfaces and areas to be sodded, should be seeded, fertilized, and mulched.

3. **Sodding.** Where developed properties or areas of intensive mowing abut the project, each areas disturbed by construction should be sodded and watered sufficiently to establish growth.

The INDOT *Standard Specifications* and Chapter 17 provide additional details on turf establishment.

51-8.04(04) Wildlife-Habitat Replacement

To some extent, existing wildlife habitat will be disturbed due to project work. Wildlife habitats may include woodlands, overgrown fields, and pastures and wetlands. The Department’s policy is to replace any disturbed wetland. This will often require the purchase of additional right of way. To determine the project’s effect on plants and animals, the designer should review the Design and Location Study Report or, where provided, the Environmental Impact Statement or Environmental Assessment. These reports may also provide recommendations on the type and quantities of habitat to be replaced.

The designer is responsible for incorporating the mitigation of the wildlife habitat into the plans. This may include revegetation with special grasses and woody species, wetlands grading, seed mixtures, etc. However, wetland revegetation with aquatic and woody species is usually administered in a separate contract once the plans have been completed. The Office of
Environmental Services will assist in coordinating habitat types and quantities. The Services and Cultural Resources Team will assist in the development of plans and specifications.
51-9.0 SOUND BARRIER

A sound barrier is designed and erected to reduce the sound level of traffic adjacent to existing properties to an acceptable level as determined by Federal guidelines. A barrier is considered the most practical option to reduce sound when compared to other mitigating options (e.g., wider buffer zone, reducing speed, eliminating or restricting traffic or vehicular types). The Office of Environmental Services is responsible for determining the longitudinal limits of the barrier, the lateral location from the roadway, and the required height. The designer is responsible for the type selection, design of the sound barrier, and evaluating the impacts of the sound barrier on the highway design and complying with the project intent of the Office of Environmental Services.

51-9.01 Types

An absorptive or reflective sound barrier is effective in reducing the environmental impact of noise from the highway. The sound-barrier types that may be used are as follows.

1. **Earth Berm.** An earth berm is a graded mound of soil which redirects the highway sound from nearby sensitive areas.

2. **Masonry Wall.** A masonry wall is constructed from concrete blocks or bricks. Very pleasing architectural designs can be developed with this type of wall.

3. **Concrete Wall.** A concrete wall may be poured in place or precast. The advantage of a concrete wall is that decorative designs can be added to the face of the wall.

4. **Wood Wall.** A wood wall is less costly than a masonry or concrete wall and is often preferred by local residents. However, its life expectancy is typically less than that of a masonry or concrete wall.

5. **Metal Wall.** A metal wall is constructed using galvanized or treated steel panels. Concerns relative to cost and corrosion have generally limited the use of steel walls.

6. **Other Materials.** New sound barrier materials are continuously being developed, such as recycled plastic, fiberglass, composites, etc. Prior to their use, they should be reviewed by the New Products Evaluation Committee to ensure that each will meet INDOT criteria.

7. **Combination Wall.** This type uses a combination of an earth berm and one of the other material types. A combination wall is used to reduce the height of another wall type and for aesthetic purposes.
51-9.02 Design

1. **Line of Sight.** Noise waves travel in a straight line. A barrier which breaks the line of sight between the source and receiver will provide some attenuation. For roadway sources, the line of sight is drawn perpendicular to the roadway. The sound source for cars and medium-sized trucks is assumed to be the roadway surface and, for large trucks, it is 8 ft high. For the receiver, the line of sight is terminated at the expected ear height of the receiver (e.g., 8 ft). The designer must also consider that the receiver may be in a multi-storied building.

2. **Structural Design.** A sound barrier should either be in accordance with the AASHTO Standard Specifications for Highway Bridges or the AASHTO Guide Specifications for Structural Design of Sound Barriers. See Chapter 73.

3. **Length.** To block the roadway noise from the sides, the ends of the barrier should exceed the receiver by four times the distance from the barrier to the receiver; see Figure 51-9A, Sound-Barrier Placement, detail (a).

4. **Location.** Moving the barrier closer to the receiver or source will increase the effectiveness of the barrier.

5. **Gap.** A gap in the barrier for pedestrian access, cross-streets, or maintenance purposes can compromise the barrier performance. Where practical, the effects of a gap should be minimized by providing tight-fitting access doors, curving the ends of the barrier to shield nearby receivers, or overlapping sections of barrier. Figure 51-9A detail (b) illustrates the minimum distance required to maintain the acoustical effectiveness of the wall for overlapping barriers.

6. **Right of Way.** Additional right of way may be required for the installation and maintenance of the sound barrier.

7. **Roadside Safety.**
   a. **Clear Zone.** Section 49-2.0 provides the Department’s design criteria for clear zone. If practical, a sound barrier should be placed outside of the clear zone. If the barrier is within the clear zone, an integral concrete barrier shape or a metal barrier rail should be considered to shield a run-off-the-road vehicle from the barrier.
   b. **Terminal.** A sound barrier should be terminated outside the clear zone. However, if the end of the barrier is within the clear zone, the designer should consider
protecting the end with guardrail or an appropriate impact attenuator. Section 49-6.0 discusses the design of impact attenuators.

c. Traversability. If the sound barrier is an earth berm, the toe of the barrier should be traversable by a run-off-the-road vehicle (see Section 49-3.02).

d. Protrusion. A protrusion may become a safety hazard if it is struck or is dislodged by a vehicle. Figure 51-9B, Sound-Barrier Protrusions, illustrates the preferred practice for placing barrier protrusions and decorative facing.

8. Emergency Access. Where sound barriers are placed relatively close to the roadway (e.g., at the edge of shoulder), sufficient escape routes must be provided in the wall to allow individuals to quickly leave the roadway in an emergency. These escape routes may be provided by inserting doors or overlapping walls. Item 5 above discusses the preferred methods for providing gaps in the barrier design. Where provided, access to fire hydrants should also be incorporated into the wall design.

9. Sight Distance.

   a. At-Grade Intersection. A sound barrier should not be located in the triangle required for intersection sight distance. Section 46-10.0 provides the criteria to determine the required sight-distance triangle.

   b. Entrance Ramp. A sound barrier should not block the line of sight between the vehicle on a ramp and an approaching vehicle on the major roadway. Therefore, a sound barrier should not be located in the gore area between an entrance ramp and freeway mainline.

   c. Horizontal Sight Distance. A sound barrier can also restrict sight distance along the inside of a horizontal curve. Section 43-4.0 provides the criteria to determine the middle ordinate value which will yield the necessary sight distance. The location of the sound barrier should be outside this sight line.

10. Interference with Roadside Appurtenances. The proposed location of a sound barrier can interfere with proposed or existing roadside features, including signs, sign supports, utilities, or lighting facilities. The designer must determine if these features are in conflict with the sound barrier.

11. Sound Considerations. The noise reduction provided by a barrier depends upon the diffraction of sound over the top and flanking around the sides of the barrier, the
transmission of sound through the barrier, and the multiple reflection caused by double barriers. Some barrier types can absorb some of the sound energy. The contribution of this absorption depends on the barrier surface, shape, and material type. A hard, smooth surface will generally reflect the noise off the wall. If barriers are to be placed on both sides of the roadway, the designer also should consider the impact of the reflected noise on the receiver.

12. **Drainage.** Drainage may be accomplished by leaving a gap on the bottom and backfilling with gravel, by providing a hinged flap, by providing a closed drainage system, etc. The barrier’s acoustical design should be maintained (i.e., no open holes in the wall).

13. **Landscaping.** Consideration should be given to providing landscaping treatments that will enhance the aesthetics and design of a sound barrier. Plantings should be provided, where practical, both in front of and behind the barrier. Low-maintenance plantings should be used behind the wall.

14. **Aesthetics.** Appearance plays a critical role in the acceptance of the sound barrier. The barrier should either be blended into the background or made aesthetically pleasing. Various types of materials, texture, and color should be considered. Smooth surfaces are not recommended.

Due to the size of a sound barrier, the designer should strive to reduce the tunnel effect by using variations of form, wall types, and surface treatments.

From both a visual and safety standpoint, a sound barrier should not begin or end abruptly. It should be transitioned from the ground line to its full height. This can be accomplished by using earth berms, curving the wall back, sloping the wall downward, or stepping the wall down.

15. **Public Involvement.** Early community participation in the selection of various sound barrier options is encouraged to ensure community acceptance of the wall.

16. **Maintenance Considerations.** The location and design of a sound barrier should reflect the following maintenance factors.

   a. The sound barrier must be located so maintenance crews can easily access the wall for routine repairs.

   b. The sound barrier should be constructed of materials that discourage vandalism (e.g., graffiti) and allow for easy cleaning. The maintenance of barrier materials is
less costly if unpainted surfaces such as weathering steel, concrete, pressure-treated wood, or naturally weathered cedar or redwood are used.

c. The sound barrier should be designed so that damage can be easily repaired. The barrier materials should be commercially available to reduce the need for keeping large stocks of material on hand.

d. The sound barrier should be located so that other maintenance operations can be reasonably performed (e.g., mowing, light-bulb replacement, sign cleaning, spraying). If the barrier is located near the shoulder, access for maintenance behind the wall should be provided from local streets or through overlapping gaps.

e. The sound barrier should be located so that it will not impact snow removal operations. A barrier located at the edge of the shoulder will require manual removal of snow from the roadway.

51-10.0 HAZARDOUS MATERIALS

Hazardous-waste sites can impact all phases of highway activities, including project development, design, right of way, construction, and maintenance. These impacts can increase costs and delay a highway project. Ownership of a site from which there has been a release, or threat of a release of a hazardous substance, may indicate liability whether the contamination is the result of the agency’s actions or those of others.

51-10.01 Responsibility

The Office of Environmental Services is responsible for ensuring that the initial site assessment is performed during the environmental stage. If the initial site assessment and coordination with other agencies identifies the need for additional work, a consultant will be used to conduct a preliminary site assessment. The Production Management Division and Office of Real Estate will be provided with summaries or copies of the information gathered on hazardous waste by the Office of Environmental Services, typically at the time of environmental-document approval.

If high levels of contamination have been detected, the Office of Environmental Services will forward the initial site assessment and the preliminary site investigation to the appropriate section of the Indiana Department of Environmental Management (IDEM), and it will request that they become involved with the property owner to characterize the site and develop a remedial plan to
clean the site. This will be concurrent with the development of the preliminary plans. The Office of Environmental Services will monitor the progress of IDEM.

At the time of the preliminary field check, the Office of Environmental Services should be able to inform both the Production Management Division and Office of Real Estate on the status of the efforts of IDEM. At this stage, decisions can be made for the site. This may include redesigning the project to avoid the site, considering various land-acquisition strategies, or delaying or dropping the project from further development due to significant hazardous-waste considerations.

51-10.02 Location

Hazardous materials can emerge from almost anywhere. Common possible locations include abandoned or active storage tanks, oil lines, illegal dumping sites, abandoned chemical plants, service stations, paint companies, machine shops, metal processing plants, electronic facilities, dry cleaning establishments, old railroad yards, auto junkyards, landfills, or bridges with lead base paints. Early indicators of contamination include groundwater contamination of nearby wells, discarded barrels, soil discolorations, liquid discharges, odors, abnormalities in vegetation, and extensive filling and regrading. If there is a chance that a site may contain hazardous materials, the Office of Environmental Services should be contacted to determine if detailed testing of the site is warranted. If hazardous materials are suspected on a property, no attempt should be made to enter the property until the site has been cleared by IDEM.

51-10.03 Cleanup

Once the hazardous-material location is known, its location must be shown on the plans. The type of contamination, if known, must also be provided. The specifications or special provisions should include detailed instructions on the procedure for removing the material and properly disposing of the wastes. For example, on a bridge with lead-based paint, waste materials from sandblasting will not be permitted into the air or onto the ground, but instead must be collected and properly disposed.

Certain cleanup sites and materials may require a specialist contractor to determine the location and size of the contaminated site and to provide for the proper removal and disposal of the contaminated materials. The specialist contractor will be required to complete the cleanup prior to construction.
51-11.0 MAILBOXES

A mailbox or newspaper tube that is serviced by a carrier in a vehicle may constitute a safety hazard, depending upon its placement. Therefore, the designer should make every reasonable effort to replace all each non-conforming mailbox with one that is in accordance with the INDOT Standard Drawings series 611-MBAS and the AASHTO A Guide for Erecting Mailboxes on Highways. Removal and replacement of a mailbox can be a sensitive issue and should be reviewed with the postage patron prior to its removal or replacement.

51-11.01 Location

A mailbox should be placed for maximum convenience to its patron, consistent with safety considerations for highway traffic, the carrier, and the patron. Consideration should be given to the minimum walking distance in advance of the mailbox site and possible restrictions to intersection sight distance at an intersection or drive entrance. A new installation should, where feasible, be located on the far right side of an intersection with a public road or drive entrance.

A box should be placed only on the right-hand side of the highway in the direction of travel of the carrier, except on a one-way street where it may be placed on the left-hand side. It is undesirable to require pedestrian travel along the shoulder. However, this may be the preferred solution for a distance of up to 200 ft when compared to constructing a turnout in a deep cut, placing a mailbox just beyond a sharp crest vertical curve with poor sight distance, or constructing two or more closely-spaced turnouts.

Placing a mailbox along a high-speed, high-volume highway should be avoided if other practical locations are available. A mailbox should not be located where access is from a freeway or where access, stopping, or parking is otherwise prohibited by law or regulation. A mailbox should not be at a location that would require a patron to cross the lanes of a divided highway to deposit or retrieve mail.

Placing a mail stop near an intersection will have an effect on the operation of the intersection. The nature and magnitude of this impact depends on traffic speed and volume on each of the intersecting roadways, the number of mailboxes at the stop, type of traffic control, how the stop is located relative to the traffic control, and the distance the stop is from the intersection. The INDOT Standard Drawings series 610-MBAP show the possible location of a mail stop at a rural intersection.

A mailbox should be located such that a vehicle stopped adjacent to it is clear of the adjacent traveled way. This need not apply to a low-volume, low-speed street or road. However, a vehicle
stopped at a mailbox should be clear of the travelway. The higher the traffic volume or speed, the
greater the clearance should be. Figure 51-11A provides guidelines for the lateral placement of a
mailbox.

A mailbox approach should be provided if a useable shoulder of 10 ft or wider is unavailable. The
INDOT Standard Drawings series 610-MBAP provide additional details for the design of an
approach for a mail stop.

51-11.02 Design

The INDOT Standard Drawings provide the design criteria for the proper placement of a mailbox.
The designer should also consider the following.

1. **Height.** A mailbox is located such that the bottom of the box is 3 ft to 4 ft above the mail-
stop surface.

2. **Multiple Mailboxes.** To reduce the possibility of ramping, multiple mailboxes should be
separated by a distance of at least three-fourths of their height above the ground.

4. **Neighborhood Delivery and Collection Box Unit.** This consists of a cluster of 8 to 16
locked boxes mounted on a pedestal or within a framework. One cluster can weigh from
100 lbs to 200 lbs and may be a roadside hazard. It should be located outside the clear zone
or only on a low-speed curbed facility. It is located in a trailer park, apartment complex, or
new residential subdivision.

51-12.0 ROUNDABOUTS [Rev. APR. 2013]

51-12.01 Introduction

This Section is intended to assist the designer in the study, design, and construction of a modern
roundabout. The principles and direction identified herein should be used for each roundabout
being planned, designed, or constructed on an INDOT-maintained route. This Section is a
supplement to FHWA RD-00-067, *Roundabouts: An Informational Guide*, which is available from
http://www.tfhrc.gov/safety/00068.htm. Throughout this Section, such document is
identified as FHWA Roundabout Guide, and is cited. Other supplemental information is
provided where relevant.
This Section is not intended to serve as a comprehensive and rigid set of design standards. Rather, it is intended to provide general guidance and identify considerations related to some roundabout-design issues. The designer should recognize that roundabout design does not entail a strict pre-defined process with repeated application of the same rules at each intersection considered. Instead, it requires the judicious application of roundabout design principles that help to reach the optimal geometric design at each individual location. This process often involves trade-offs between competing objectives to reach the best solution. Roundabout design requires the use of engineering judgment and thought on the part of the designer. The use of sound engineering principles and common sense are also vital to successful roundabout design. The use of this Section does not relieve the designer of his or her personal responsibility to produce a design for a roundabout that functions safely and efficiently within the context of a given location. This Section does not address all of the specific situations which can arise during the course of roundabout design. If a unique situation arises, the designer should see the FHWA Roundabout Guide and contact experienced roundabout designers.

This Section includes information regarding roundabout planning, safety, geometric design, pavement markings, lighting, landscaping, signage, public involvement, and other design or operational considerations. For each topic or issue, guidance is provided regarding goals and objectives, the location of relevant available standards, and related factors that should be considered. Rather than repeat information that is included elsewhere, e.g., the FHWA Roundabout Guide, the MUTCD, AASHTO Policy on Geometric Design of Highways and Streets, etc., references are provided so that information can be obtained from each source.

### 51-12.02 Definitions

The following definitions will apply.

1. **Approach Design Speed.** The design speed of the roadway leading into the roundabout.

2. **Bicycle Treatment.** This provides a bicyclist the option of traveling through the roundabout either as a vehicle operator or by using a shared-use path around the exterior of the intersection.

3. **Bypass Lane.** A lane that separates right-turn movements from the roundabout-circulating traffic (see Figure 51-12P).

4. **Central Island.** The area of the roundabout inside the circulatory roadway including the truck apron.
5. **Central-Island Diameter.** The diameter of the central island, including the truck apron (see Figure 51-12A).

6. **Circulatory Lane.** A lane used by vehicles circulating in the roundabout.

7. **Circulatory Roadway.** The travel lanes adjacent to the central island and outside the truck apron, including the entire circumference of the circle.

8. **Circulatory-Roadway Width.** The width between the outer edge of the inscribed diameter at the curb face of this roadway and the central island curb face. It is typically 1 to 1.2 times the width of the widest entry width. It does not include the width of a traversable apron, which is defined to be part of the central island. The circulatory roadway width defines the roadway width, curb face to curb face, for vehicle circulation around the central island (see Figure 51-12A).

9. **Conflict Point.** A point where traffic streams cross, merge, or diverge.

10. **Deflection.** The change in the path of a vehicle imposed by the geometric features of a roundabout, resulting in a slowing of vehicles (see Figure 51-12JJ).

11. **Entry Angle.** The angle between the entry roadway and the circulating roadway, measured at the yield line. See Ourston Roundabout Engineering.

12. **Entry Lane.** The lane or set of lanes for traffic approaching the roundabout (see Figure 51-12A).

13. **Entry Radius.** The radius of curvature of the outside curb face at the exit.

14. **Entry Width.** The width of the roadway where it enters the roundabout. It is measured perpendicularly from the outside curb face to the inside curb face at the splitter-island point nearest the inscribed circle.

15. **Exit Lane.** The lane or set of lanes for traffic leaving the roundabout (see Figure 51-12A).

16. **Exit Radius.** The radius of curvature of the outside curb at the exit.

17. **Exit Width.** This defines the width of the exit where it meets the inscribed circle. It is measured perpendicularly from the right curb-face edge of the exit to the intersection point of the left curb-face edge and the inscribed circle (see Figure 51-12A).
18. **Fastest Path.** The shortest possible route that a single vehicle can travel through a roundabout in the absence of other traffic, and ignoring all lane markings. The fastest path determines the fastest possible entering, exiting, and circulating speeds within a roundabout.

19. **Flare Length.** The distance over which the approach roadway widens to the entry width, if such flaring is present.

20. **Half Width, \(V\).** The width of the existing approach roadway before it starts to widen, if flaring is present.

21. **Inscribed Circle.** The outer edge of the circulatory roadway.

22. **Inscribed-Circle Diameter, ICD.** The outside diameter of the inscribed circle measured from face of curb to face of curb (see Figure 51-12A).

23. **Landscaping Buffer.** Often provided to separate vehicular and pedestrian traffic and to encourage pedestrians to cross only at the designated crossing locations. A landscaping buffer can also improve the aesthetics of the intersection.

24. **Natural Vehicle Path.** The path that a driver will navigate a vehicle given the layout of the intersection and the ultimate destination.

25. **Path Overlap.** This occurs where the natural path of a vehicle traveling through a roundabout overlaps the path of an adjacent vehicle. See the FHWA *Roundabout Guide*, Section 6.4.2.

26. **Pedestrian Crossing.** This is typically located about 10 ft before the yield line and is usually a painted crosswalk. A pedestrian crossing allows a pedestrian to cross in one direction of vehicle travel at a time and provides median refuge in the splitter island.

27. **Roundabout.** A circular at-grade intersection with yield control of all entering traffic, channelized approaches with raised splitter islands, counterclockwise circulation, and appropriate geometric curvature.

28. **Splitter Island.** The raised island at each two-way leg between entering and exiting vehicles, designed primarily to control entry and exit speeds by providing deflection. It also prevents wrong-way movements and provides pedestrian refuge.

29. **Truck Apron.** The paved portion of the central island located adjacent to the circulating roadway. It is defined by a sloping curb on the outside and helps accommodate large trucks.
30. **Yield-at-Entry.** The requirement that vehicles on all entry lanes must yield to vehicles within the circulatory roadway.

31. **Yield Line.** A pavement marking used to mark the point of entry from an approach into the circulatory roadway and is marked along the inscribed circle.

### 51-12.03 Roundabout Types

#### 51-12.03(01) Mini-Roundabout

A mini-roundabout is a small roundabout used in an urban environment, with speeds of 30 mph or lower. Figure 51-12B provides an example of a typical mini-roundabout. It can be useful where conventional roundabout design is precluded due to right-of-way constraints. In a retrofit application, a mini-roundabout is relatively inexpensive because it typically requires minimal additional pavement at the intersecting roads, such as minor widening at the corner curbs. It should be used where there is insufficient right of way for an urban compact roundabout.

Because it is small, it is perceived as pedestrian-friendly with short crossing distances and low vehicle speeds on approaches and exits. The mini-roundabout is designed to accommodate passenger cars without requiring them to be driven over the central island consisting of a dome of asphalt painted white. To maintain its perceived compactness and low-speed characteristics, the yield lines are positioned just outside the swept path of the largest expected vehicle. However, the central island is mountable, and larger vehicles can cross over the central island, but not to the left of it. Speed control around the mountable central island should be provided by means of horizontal deflection. Capacity for this type of roundabout is expected to be similar to that of the urban compact roundabout. For design assistance, see Mini-Roundabouts, A Definitive Guide by Clive Sawers.

#### 51-12.03(02) Urban Compact Roundabout

Like a mini-roundabout, this roundabout type is intended to be pedestrian- and bicyclist-friendly because its perpendicular approach legs require low vehicle speed to make a distinct right turn into and out of the circulatory roadway. All legs have single-lane entries. However, the urban compact treatment satisfies all of the design requirements of an effective roundabout.

The geometric design includes raised splitter islands that incorporate at-grade pedestrian storage areas, and a non-mountable central island. There is an apron surrounding the non-mountable part of the compact central island to accommodate larger vehicles. Figure 51-12C provides an example of a typical urban compact roundabout.
51-12.03(03) Single-Lane Roundabout

A single-lane roundabout has single-lane entries at all legs and one circulating lane. It has non-mountable raised splitter islands, a mountable truck apron, and a non-mountable central island (see Figure 51-12D). Right-turn bypass/slip lanes can be added as required.

51-12.03(04) Multilane Roundabout

A multilane roundabout has at least one entry or exit with two or more lanes and more than one circulatory lane (see Figures 51-12E and 51-12F). To balance the needs of passenger cars and trucks and to provide safety, trucks should encroach on adjacent lanes within the circulating roadway. However, the district or local public agency’s preference should be sought prior to assuming truck encroachment on adjacent lanes for design purposes.

51-12.03(05) Teardrop Roundabout

A teardrop roundabout is used at an interchange. See Figure 51-12G for possible configurations of such a roundabout.

51-12.04 Planning

51-12.04(01) Introduction

A roundabout should be considered as one potential intersection option within an INDOT-sponsored or -funded planning study or project since it offers improved safety, cost savings, and enhanced traffic operations. This includes a proposed freeway interchange where an at-grade intersection currently exists or will be created at the ramp terminals. A comparison of roundabout practicality or feasibility versus other intersection types should be conducted, considering safety, traffic operations, capacity, right-of-way impacts, and cost. Other factors as described below can also be included in the evaluation if desired and deemed appropriate. In conducting such comparisons, a roundabout is not always the optimal solution, but it can often offer significant benefits.

51-12.04(02) Planning Process

The typical planning process includes consideration of the following:
1. data collection including recent adverse-accident history and types of crashes;
2. development of 20-year traffic projections;
3. capacity analysis to analyze traffic operations and geometry;
4. preparation of a roundabout-concept design;
5. public involvement;
6. comparison to other intersection types including the do-nothing alternative;
7. documentation via a report or memorandum;
8. selection of preferred option; and
9. analysis of causes of a large number of crashes or potential for them.

The goal of the planning process is to make a sound decision regarding whether a roundabout is feasible, whether it is a better solution than other intersection types, and whether it should be advanced into the preliminary design phase. Early and ongoing coordination with the Production Management Division’s Office of Roadway Services, the Planning Division’s Safety Team, or the applicable local public agency should be carried out throughout the duration of the project at key milestones. The FHWA Roundabout Guide, Chapters 2 and 3, provide additional information regarding planning for a roundabout.

51-12.04(03) Required Data

Data that is typically required in order to evaluate a roundabout includes the following:

1. existing morning and afternoon peak-hour turning-movement counts;
2. major traffic generators, if present, with shift changes that occur during off-peak hours;
3. INDOT-approved design-year morning and afternoon peak-hour turning-movement projections;
4. design vehicle to be accommodated;
5. base mapping, either aerial photograph, aerial mapping, or survey;
6. right-of-way mapping;
7. crash data for the most recent three-year period available, though 5 years is preferred;
8. location of nearby intersections and signal timing information, if applicable;
9. location of major constraints near the intersection, i.e., right of way, major utilities, structures, railroad crossings, bodies of water;
10. existing and future planned bicycle and pedestrian facilities;
11. truck percentages; and
12. accommodation of disabled persons.

Data that is desirable to obtain, though not necessarily required for each situation, includes the following:
1. existing pedestrian counts;
2. previously prepared construction plans or as-built plans showing the existing intersection; and
3. utilities information.

51-12.04(04) Evaluation Criteria

In assessing the desirability of a roundabout relative to other intersection types, evaluation criteria should include the following:

1. safety;
2. capacity;
3. traffic operations;
4. cost;
5. design life of 20 years; less than this is undesirable;
6. right-of-way impacts;
7. safe accommodation of pedestrians and bicyclists;
8. aesthetics;
9. proximity to other intersections;
10. drive accommodation and access-management opportunities;
11. public input;
12. constructability;
13. traffic maintenance; or
14. social, economic, noise, and environmental impacts.

51-12.04(05) Capacity Limitations

As discussed further in Section 51-12.05, a roundabout’s capacity is determined based on its geometry and peak-hour traffic volume and turning patterns. Because geometry and peak-hour traffic volume can vary considerably within a single-lane, two-lane, or three-lane roundabout, it is not possible to develop a precise capacity that applies to each category. However, Figures 51-12H and 51-12 I provide approximate maximum daily and hourly service-volume capacities, respectively, for each category. The capacities provided in Figures 51-12H and 51-12 I are only a general guide. There is no substitute for an intersection-specific capacity analysis.

51-12.04(06) Beneficial Location and Applications
Implementation of a roundabout can be beneficial to the traveling public in a number of situations. The following identifies some of the most common locations or applications where installation of a roundabout can be advantageous. However, the designer or other decision-maker should recognize that this list is general and will not apply to every situation. There are useful applications of a roundabout that are not included below. The applications shown below may not always be appropriate. Site-specific analysis of roundabout feasibility should be conducted at each individual location, as follows.

1. **High-Speed Rural Intersection.** Studies and experience show that a roundabout is an exceptional safety countermeasure at this type of location. Other states that have installed roundabouts at such locations have reported reductions in total crashes, injury crashes, and fatal crashes. This is consistent with the experiences of other countries.

2. **Intersection with Crash History.** Studies and experience show that a roundabout can provide reductions in injury crashes and fatal crashes. The specific types of crashes which can be reduced include left-turn head-on and angled crashes.

3. **Intersection with Traffic-Operational Problems.** A properly designed roundabout can be effective in eliminating congestion and delays.

4. **Closely-Spaced Intersections.** A roundabout can eliminate traffic queuing from one intersection into another. It can also eliminate problems related to coordination of traffic-signal timing between closely-spaced intersections.

5. **Intersection Near a Structure.** A roundabout most often does not require as many approach lanes as a signalized intersection for vehicle storage. Where a bridge structure is located near an intersection, installing a roundabout can allow the use of a shorter or narrower bridge structure, resulting in significant cost savings. The most common situation is at a freeway interchange.

6. **Freeway Interchange.** A roundabout can be beneficial at the ramp terminals of a freeway interchange. Random spacing of vehicles exiting a roundabout can be beneficial as they merge from an on-ramp into the stream of traffic of a freeway mainline. This is similar to the effect achieved through ramp metering in a congested urban area.

7. **As a Part of an Access-Management Program.** Since a roundabout can accommodate U-turns, it can be implemented as a part of an overall access management plan, especially at an intersection that displays other characteristics that make a roundabout desirable, such as crash problems or traffic-operational problems. For this situation, a roundabout can function as a median turnaround.
8. **Intersection with Unusual Geometry.** Since roundabout geometry is relatively flexible, an intersection with unusual geometrics can be improved with the installation of a roundabout.

9. **Intersection at a Gateway or Entry Point to a Campus, Neighborhood, Commercial development, or Urban Area.**

10. **Intersection where Community Enhancement May Be Desirable.**

11. **Intersection Near a School.**

51-12.04(07) Non-Beneficial Locations and Applications

There are locations or applications where a roundabout may not be beneficial. The listing provided below is general and will not apply to each situation. A roundabout may still be a beneficial solution though the location includes some of the characteristics listed below. Each situation should be independently analyzed. Some of the circumstances listed below cause potential concerns at a traffic-signalized intersection.

1. **Intersection Within a System of Coordinated Traffic Signals.** If a corridor includes multiple traffic signals with a functioning progression, a roundabout may not be the best overall solution. For this situation, a roundabout can result in the disruption of traffic platoons.

2. **Intersection with Steep Grade.** It is undesirable to construct a roundabout where the grade through the intersection is steeper than 5%. Potential concerns include the ability for a driver to stop where the road is snow-covered or icy, or the potential for a truck to tip over. This applies a roundabout and an intersection with traffic signals. A roundabout can be constructed where the approach grades are steeper than 5% if the approaches’ grades within about 100 ft of their intersection with the roundabout are 5% or flatter.

3. **Intersection Where Stopping Sight Distance Cannot Be Achieved.** A driver should be able to identify an intersection as such, and have adequate time to stop if necessary. This concern is due to either a vertical or horizontal curve, with or without superelevation.

4. **Intersection Near Railroad Crossing.** If a roundabout is being considered near a railroad crossing, it should be designed to ensure that traffic will not queue from the roundabout onto the railroad tracks or vice versa. Traffic queuing from a railroad crossing into the circulatory roadway of a roundabout can result in gridlock with the result being that motorists cannot enter or exit the roundabout from any direction.
5. Closely-Spaced Intersections. A roundabout should be designed to ensure that traffic in adjacent intersections will not back up into a roundabout. This should be considered where a roundabout is installed to mitigate an existing congested intersection. In this situation, the roundabout can usually process traffic more efficiently than the previous intersection, with the result being that traffic in downstream intersections can back up into the roundabout.

51-12.04(08) **Comparison of Roundabout Categories**

Figures 51-12J and 51-12K summarize and compare the fundamental design and operational elements for each roundabout category. The following provides a qualitative discussion of each category.

51-12.05 **Roundabout Operation**

51-12.05(01) **Introduction**

A roundabout brings together conflicting traffic streams and allows the streams to safely cross paths, traverse the roundabout, and exit to their desired directions at reduced speeds. A modern roundabout does not have merging or weaving between conflicting traffic streams. Compactness of circle size and geometric speed control make it possible to establish priority to circulating traffic. The geometric elements of the roundabout reinforce the rule of circulating traffic priority and provide guidance to drivers approaching, entering, and traveling through a roundabout.

Drivers approaching a roundabout must slow to a speed that will allow them to safely interact with other users and negotiate the roundabout. The width of the approach roadway, the curvature of the roadway, and the volume of traffic present on the approach govern this speed. As drivers approach the yield point, they must first yield to pedestrians and then to conflicting vehicles already in the circulatory roadway. The widths of the approach roadway and entry determine the number of vehicle streams that can form side-by-side at the yield point and influence the rate at which vehicles can enter the circulatory roadway. The size of the inscribed circle affects the radius of the driver’s path, which in turn determines the speed at which the driver travels in the roundabout. The width of the circulatory roadway determines the number of vehicles that can travel side-by-side in the roundabout.

A capacity analysis is required prior to concept design. To determine the required geometry and corresponding queues and delays for a roundabout, a capacity analysis should be conducted. INDOT does not mandate use of a particular software for capacity analysis. There are a number of
roundabout-capacity-analysis softwares available such as RODEL, SIDRA, ARCADY, VISSIM, PARAMICS, etc. The designer has the discretion to utilize the most appropriate software for analysis based on his or her own research and knowledge. Unless otherwise directed, INDOT-approved 20-year traffic projections for morning and afternoon peak hours should be used for the capacity analysis.

In conducting a capacity analysis, geometric parameters may require adjustment through an iterative process, i.e., numerous adjustments, to achieve the desired delays and LOS in each peak hour. During this optimization process, site constraints should be considered, such as major issues with regard to right of way, nearby bridges, utilities, etc., and other roundabout-design principles related to speed control.

51-12.05(02) Operational-Analysis Tools

A roundabout-intersection analysis model can be empirical or analytical. An empirical model relies on field data to develop relationships between geometric design features and performance measures such as capacity and delay. An analytical model is based on the concept of gap-acceptance theory. Extensive research by TRL conducted in England supports the empirical-formula method over the gap-acceptance method. RODEL is a software program that is based on this research and the empirical-formula method. RODEL permits expedient testing of what-if scenarios, thus allowing optimization of design rather than the one that satisfies minimum criteria. Small changes in roundabout geometry such as entry width or flare length may increase the probability that the roundabout will perform well at a high v/c ratio. ARCADY is another empirical software. Softwares such as VISSIM and SIDRA are based on gap-acceptance techniques.

51-12.05(03) Single-Lane Roundabout Entry Capacity

Roundabout capacity is site-specific, since it is related to the geometric features of each site. For planning purposes, a single-lane roundabout can be expected to handle an AADT of up to approximately 25,000 and peak-hour flow between 2,000 vph and 2,500 vph. These are the total entering volumes from all entries combined.

RODEL assumes that an entry as wide as 18 to 20 ft to accommodate trucks can represent two narrow lanes instead of one wide lane. Where only one entering and one circulatory lane are present with such a width, this can result in overprediction of capacity and underprediction of delays and queues. Therefore, a single-lane entry, if modeled using RODEL, should be evaluated for a width of 15 ft or less, with 13 to 14 ft being more conservative. This capacity analysis procedure is reasonably conservative and should be used if the actual entry geometry is designed to be wider to accommodate trucks.
51-12.05(04) **Single-Lane Exit Capacity**

It is difficult to achieve an exit flow on a single lane with a DHV of higher than 1,400 vph, under operating conditions for vehicles which include tangential alignment, and no pedestrians or bicyclists. Under normal urban conditions, the exit-lane capacity will be in the range of 1,200 vph to 1,300 vph. Therefore, exit flow exceeding 1,200 vph can indicate a lower LOS or the need for a multilane exit.

51-12.05(05) **Multilane-Roundabout Capacity**

For planning purposes, a multilane roundabout can be expected to handle an AADT of 25,000 to 55,000 and a peak-hour flow of 2,500 vph to 5,500 vph. However, peak-hour traffic for each individual location should be evaluated before final conclusions are reached. The expected capacity can be higher with the implementation of bypass lanes.

51-12.05(06) **Pedestrian and Truck Effects on Entry and Exit Capacity**

A pedestrian crossing at a marked crosswalk that has priority over entering motor vehicles can have an effect on the entry capacity. Where pedestrian volume is relatively high, the effect on capacity is assessed by using the pedestrian-capacity reduction factors shown in the FHWA Roundabout Guide, Exhibits 4-7 and 4-8. These factors should be considered in the capacity analysis. For example, these can be entered into the capacity factor field in using RODEL for each leg of the roundabout. A similar concern can occur at the roundabout exit where pedestrians cross that cannot be modeled in RODEL. The use of microsimulation models should be considered where high pedestrian volume is anticipated. Microsimulation models are further discussed in Section 51-12.05(14).

High truck volume can also reduce roundabout capacity and should be considered in the analysis. In using RODEL, truck percentage should be represented by modifying the Passenger Car Units (PCU) field.
51-12.05(07) Key Roundabout Parameters Affecting Operating Capacity

The key roundabout-design parameters are shown in Figure 51-12L and defined in Figure 51-12M.

Figure 51-12N shows typical relationships between the six geometric design parameters and roundabout capacity. Figure 51-12O shows that the inscribed-circle diameter has less impact on roundabout capacity than other geometric parameters. These figures are shown for reference only and are based on the capacity results using RODEL software. Other softwares may depict different results.

Research indicates that approach width, entry width, effective flare length, and entry angle have the most significant effect on entry capacity. Where circulating flow is high, increasing the inscribed-circle diameter (ICD) will also substantially increase capacity. Figure 51-12O shows that the capacity on one leg of the roundabout is increased by 401 vph if the ICD is increased from 130 to 195 ft. This increased capacity can occur on more than one leg.

The entry radius has little effect on capacity provided that it is 65 ft or longer. Using an entry radius shorter than 45 ft reduces capacity with increasing severity. A small entry radius tends to produce a large entry angle and vice versa. A perpendicular entry of 70 deg or greater, and an entry radius of less than 50 ft will reduce capacity. Thus, the geometric elements of a roundabout, together with the volume of traffic desiring to use a roundabout at a given time, determine the efficiency of the roundabout’s operation.

51-12.05(08) Lane Balance

Lane balance and utilization is tested at a multilane roundabout for both peak hours after the geometry has initially been identified. Incorrect lane assignments, i.e., right, through, left, will sometimes affect lane utilization enough to result in significant unbalanced lane use, long delays, and long queues. Therefore, once roundabout geometry is identified at a multilane roundabout, lane usage should be analyzed in performing the capacity analysis. For example, this can be done by manipulating the capacity factor function in the RODEL software. This will result in identification of proper lane assignments and should be reflected in the concept design.

In using RODEL software, the user should toggle from the flow factor to the capacity factor to test lane balance and identify lane assignments. Once the capacity factor has been enabled, this value should be changed from the default 1.00 to 0.50 for a two-lane entry, or 0.33 for a three-lane entry, for the leg to be analyzed. This allows the capacity of one lane to be tested with the peak-hour traffic volume for a specific turning movement, i.e., right, through, left. The movement to be analyzed should be isolated by zeroing out the other two movements. If the
predicted queues and delays for the movement are acceptable using one lane, the lane can either assigned only for that movement, e.g., left only, right only, etc., or as a combined use which includes that movement, e.g., left or through, etc. More than one lane may be needed for the movement, e.g., double left, etc., if queues and delays are not acceptable. This process can be repeated for each movement and each leg to determine lane assignments for the intersection. Based on these results, the geometry and pavement markings can be adjusted. Other software can also include the provision to evaluate the lane balance and lane assignments, which is recommended for multilane-roundabout analysis.

51-12.05(09)  Bypass Lane

A bypass lane allows vehicles to circumvent a roundabout, providing additional capacity. It is used where a high percentage of turning movements are right turns. A bypass lane should be used only if other geometric layouts fail to provide acceptable traffic operations. The decision to use a bypass lane should consider pedestrian and right-of-way constraints. A bypass lane can provide significant benefits. The types of bypass lanes are a free-flow lane which allows vehicles to bypass the roundabout and then merge into the exiting stream of traffic, or a semi-bypass lane which requires approaching vehicles to yield to traffic leaving the adjacent exit. For more information, see Section 51-12.09(14) and Figure 51-12P.

51-12.05(10)  Peak-Hour Factor

A peak-hour factor, or flow ratio in the RODEL software, should be considered for capacity analysis and for calculating queues and delays. This factor represents the rise and fall of traffic during a peak hour, which can impact the roundabout’s capacity and operations.

51-12.05(11)  Diameter

The ICD used for the capacity analysis must satisfy speed-control criteria, as further discussed below.

51-12.05(12)  Effective Width

The geometry used for capacity analysis should be measured curb face to curb face, or effective width.
51-12.05(13) Simulation Tools

A number of simulation tools are available to visualize the operation of a roundabout that are helpful for a complex situation. The purpose of performing a microsimulation is for visualization and to provide the ability to visualize multiple intersection operations along an arterial with integrated signalized, stop-controlled, and roundabout intersections. Simulation tools can be effective for showing general roundabout traffic operations to the public.

51-12.05(14) Entry Width

The range of design values for roundabout geometrics are shown in Figure 51-12J. These values are intended for general guidance only, as each roundabout design is unique with respect to location, design criteria, traffic flow, and other site specifics.

1. Roundabout Performance Measures. The measures that are used to estimate the performance of a roundabout design are delay and queue length. Each measure provides a unique perspective on the quality of service of a roundabout under a given set of traffic and geometric conditions.

Delay is a standard parameter used to measure the performance of an intersection or approach. The Highway Capacity Manual identifies delay as the primary measure of effectiveness for both signalized and un-signalized intersections, with level of service determined from the delay estimate.

Queue length is used in assessing the adequacy of the geometric design of the roundabout approaches. The approach roadway should have adequate storage capacity so that the queue does not obstruct driveway access or another intersection. Depending on location, a queue of 10 vehicles may be unacceptable at one site while a queue of 50 vehicles at another site may be acceptable.

2. Volume Diagram and Lane Configuration Sketch. Use Figure 51-12Q to determine traffic volume; existing peak-hour turning volume for the morning, afternoon, or weekend; and design-year peak-hour turning volume. Compare the design-year flow with existing flow and check for anomalies. The design-year flow should not exceed the capacity of the surrounding network. Figure 51-12Q provides a format for a 3 or 4-leg intersection, or interchange ramp with a roundabout. Place the existing or projected peak-hour traffic volume, by movement, where indicated on the spreadsheet and it will calculate the circulating-traffic volume in the circulatory roadway adjacent to each splitter island, the exit volume, and entrance volume. Circulating flow will be shown in the boxes in the center of the diagram. These are used in the initial analysis of the roundabout. The
spreadsheet will also provide the correct input placement and values for performing the capacity analysis.

U-turn traffic will be 1% of the entering traffic volume and can be much greater where there is no median opening between roundabouts. The U-turn volume should be included in the traffic analysis.

A lane configuration for each entry should accompany the volume diagram to facilitate the selection of the number of lanes and the lane assignments. This step precedes the roundabout capacity analysis and the layout process since it affects the geometry. In Figure 51-12R, the assessment of lane assignments for Leg 1 can include three different options.

Depending on the option, a spiral marking treatment to spiral out the westbound left turn may be required. Spiral markings are discussed in Section 51-12.10(02) item 3. The southbound exit may need to become a single lane. Option 1 is the preferred and simplified lane configuration that works for both peak and off-peak periods. Figure 51-12S shows an example of the final roundabout layout.

51-12.05(15) Operations and Entry-Lane Pavement Markings

A multilane roundabout should include entry-lane markings. These markings have the potential to slightly increase capacity and decrease delays or queues. However, these variations in capacity are relatively minor and are not quantifiable. For this reason, the geometry should not be changed based on this assumed increase in capacity. The correct use of lane arrows can be beneficial to help approaching traffic achieve a desirable distribution of traffic between lanes. Inappropriate use of lane arrows can also reduce capacity, if placed incorrectly.

The reduction in capacity arising from the incorrect use of lane arrows can be severe if a high proportion of the approach volume uses one exit. For example, assume an approach on a 4-leg roundabout has three lanes, with arrows pointing left, straight, and right. If 60% of the approach flow is straight ahead, it is constrained to the middle lane, which only has one third of the approach capacity. The resulting queues can quickly expand beyond the beginning of the flare preventing access to the left and right turn lanes, further reducing capacity.

The use of appropriate lane arrows should encourage balanced lane use, thus improving capacity. Traffic often has a bias towards the right-most lane. Lane arrows can either encourage this bias, or can encourage lane balance. Figure 51-12S shows an alternative pavement marking. The best approach markings will depend on the turning volume. The markings that produce the most balanced lane utilization are preferred. The configuration shown in Figure 51-12T diagram (a)
should be utilized for frequent right-turn and through movements. The configuration shown in Figure 51-12T diagram (b) should be utilized for frequent left-turn and through movements. Lane arrows can produce subtle problems that can reduce capacity and cause accidents. The design of pavement markings is further discussed in Section 51-12.10(02).

51-12.06 Roundabout Safety

51-12.06(01) Introduction

Section 51-12.09 provides information regarding geometric design including information regarding sight distance, grades, cross slopes, etc. Following the guidance provided therein ensures the safest possible geometric design. The FHWA Roundabout Guide, Chapters 5 and 6, provides additional information regarding roundabout safety.

The principles regarding geometry that will maximize roundabout safety are as follows.

1. Minimize entry and circulatory-roadway widths, inscribed-circle diameter (ICD), and number of lanes.

2. Keep entry and exit radii within the appropriate range. Therefore, avoid very small exit and entry radii and very large entry radii.

3. Vehicle speeds should be within an acceptable range based on roundabout type along the fastest path prior to the yield line. This is further discussed in Section 51-12.09 and the FHWA Roundabout Guide, Chapter 6.

4. Keep the entry angle for each entrance between 20 deg and 30 deg.

5. Where practical, increase capacity by using a longer flare length as opposed to a wider entry.

6. Maximize the angle between adjacent legs of the intersection. This is different than the entry angle.

7. Avoid entry and exit path overlap at a multilane roundabout.

These principles should most-often apply, but there are exceptions and circumstances where they will not apply. For additional information regarding these principles and their application, see TRL Laboratory Report 1120, Accidents at 4-Arm Roundabouts, 1984.
51-12.06(02) Evaluation Process

Evaluating a roundabout as a potential safety countermeasure can be accomplished by means of a cost/benefit (C/B) ratio, or examination of existing crashes versus anticipated reductions based on roundabout-safety studies. The potential safety benefits of a roundabout can also be evaluated by means of identifying the crash frequencies and types presently occurring at an intersection and determining whether these will likely be eliminated or reduced by use of a roundabout, assuming that average crash reductions will be realized. The crash types that are of the most concern at an intersection are those that result in serious injuries and fatalities. Left turn head-on crashes and angled crashes are the most dangerous types. The potential for both of these crash types is essentially eliminated through the installation of a roundabout.

Traffic at an intersection with approach speeds higher than 45 mph is likely to experience a significant safety benefit since crash severity is often high at such an intersection. Roundabouts in other states have been shown to be an effective safety countermeasure at such an intersection. However, implementation of a roundabout should not be limited only to an intersection that experiences head-on and angled crashes, since other types of crashes are also reduced or eliminated where a roundabout is installed. In evaluating the feasibility of a roundabout, at least three years of crash data should be collected and analyzed, with five years’ worth as preferable, if available. Crash frequencies, patterns, types, and severity should be identified. Once this information is analyzed, it can be determined whether a placement of a roundabout is likely to reduce crashes.

If another type of intersection control, i.e., traffic signal, four-way stop, two-way stop, is being considered and compared to a roundabout, typical crash frequencies, severities, and rates can be estimated based on the performance of the same intersection-control type at other existing intersections in the area. Throughout the evaluation process, variables other than intersection type that are not substantially contributing to crashes should be considered.

51-12.06(03) Follow-Up Monitoring

Once a roundabout is constructed, follow-up monitoring should be conducted periodically to determine safety performance after implementation. Data for the first three to 12 months of operation should be excluded from consideration, as it should be expected that motorists are adjusting to the new intersection during this time frame. Adjustments to pavement markings and signing may be warranted based on crash patterns after implementation. The district Office of Traffic or the local public agency is responsible for monitoring the operations at a roundabout located within its service area. The procedure for conducting a before-and-after safety-benefits evaluation is included in NCHRP Report 572.
**51-12.07 Multimodal Considerations**

**51-12.07(01) Introduction**

Accommodating non-motorized users is a Department priority. Therefore, consideration should be given to non-motorized use as follows:

1. pedestrian volume is high;
2. there is a presence of young, elderly, or visually impaired citizens wanting to cross the road; and
3. pedestrians are experiencing particular difficulty in crossing and are being excessively delayed.

The adjacent land uses near a proposed roundabout location should be considered. Land use such as a school, playground, hospital, or residential neighborhood can warrant additional treatments as described below. If it is determined that bicyclist or pedestrian concerns will be a factor in the design of the roundabout, the Production Management Division’s Office of Roadway Services should be contacted for input.

**51-12.07(02) Pedestrians**

Research shown in the FHWA *Roundabout Guide* indicates that fewer pedestrian accidents with less severity are occurring at roundabout intersections when compared to signalized or unsignalized intersections with comparable volume. Relatively low-speed entries and exits should be provided to maximize pedestrian safety. Due to relatively low operating speeds of 15 to 20 mph, pedestrian safety is improved in a roundabout than in other intersection types. Figure 51-12U lists advantages and disadvantages of a roundabout as related to pedestrians.

The pedestrian crossing should be located approximately 25 ft upstream from the yield point. This helps to reduce decision-making problems for drivers. For pedestrian safety, the crossing should not be located too far from the yield line such that entering vehicle speeds are not yet sufficiently reduced or exiting vehicles are accelerating. It may be appropriate to design the pedestrian crossing at 50 to 75 ft from the yield point at a multi-lane entry. The crossing should be placed perpendicular to the direction of traffic in entrances and exits to minimize pedestrian travel and exposure time.

At a roundabout with high traffic volume or high pedestrian volume, the pedestrian crossing can be enhanced with features such as crosswalk pavement markings, colored concrete with patterned borders, lighted bollards at entries and exits, and activated push-button or automatic-detection warning signals. Where pedestrian volume is very high, consider accommodating pedestrians
with an overpass or underpass. Contact the district Office of Traffic or the local public agency in determining the appropriate pedestrian treatment.

Pedestrians are faced with the continual movement of motor traffic, and their possible inability to judge gaps in an oncoming travel stream. This is true of children, the elderly, the disabled, or the visually impaired. These pedestrians often prefer larger gaps in the traffic stream, and walk at slower speeds than other pedestrians. A pedestrian crossing should be designed in accordance with the Americans with Disabilities Act (ADA). See the FHWA Roundabout Guide, Section 5.3.3, and the MUTCD.

The pedestrian hybrid signal should be considered where there is an identified or demonstrated need to accommodate the visually impaired. This signal is currently experimental, therefore it requires a formal request from FHWA for installation.

**51-12.07(03) Bicyclists**

The operation of a bicycle through a roundabout can be a challenge to a bicyclist similar to that of a signalized intersection, especially for turning movements. As with a pedestrian, one of the difficulties in accommodating a bicyclist is the wide range of skills and comfort levels in mixing with vehicular traffic. The complexity of vehicle interactions within a roundabout can leave a cyclist vulnerable. Designated bicycle-lane markings within the circulatory roadway should not be used. A design that constrains motor vehicles to speeds more compatible with bicycle speeds is preferred.

Features such as proper entry curvature and entry widths to slow traffic entering the roundabout should be integrated into the roundabout design. The addition of a ramp from a bicycle lane to a shared-use path prior to the intersection as shown in Figure 51-12V allows a bicyclist to exit the roadway and proceed around the intersection safely through the use of crosswalks if the bicyclist is uncomfortable with mixing with motor vehicles. For additional information on bicycle-ramp design, see Section 51-12.09(21).

Bicyclists are often less visible and more vulnerable when merging into and diverging from a multilane roundabout. A wider, shared-use pedestrian-bicycle path should be provided separate from the circulatory roadway where significant bicycle volume is expected. While this will likely be more comfortable for the casual bicyclist, the experienced bicyclist will be slowed down by having to cross at the crosswalk and may choose to traverse a multilane roundabout in the same manner as a motor vehicle.
The following guidance is intended for a shared-use path at a roundabout.

1. Construct a widened sidewalk, or separate shared-use path around the outside of a roundabout to accommodate bicyclists who prefer not to travel through the roundabout.

2. Begin and end the shared-use path 50 to 150 ft upstream of the yield point to allow the bicyclist an opportunity to transition onto the path. More space may be needed if a flared entrance is provided.

3. Right-turn bypass lanes for motor vehicles may be problematic for bicyclists. The use of bypass lanes should be avoided in a high-bicycle-volume area if possible.

4. Provide a ramp or other suitable connection between the sidewalk or path and the bicycle lane, shoulders, or roadway surface on the approaching and departing roadway. The bicycle exit ramp should have a 25- to 35-deg departure angle away from the roadway. A bicycle entrance ramp should have a 25- to 35-deg angle range toward the roadway. See Figure 51-12W. The bicycle-ramp entrance should be relatively flat such that bicyclists are not directed into the travel lane for motorized vehicles, but parallel to the bicycle lane.

A grade-separation overpass or underpass for bicyclists should be considered for a high-motor-vehicle-volume roundabout also with high bicyclist volume. For information on a permanent public trail crossing a rural public road, see Section 51-7.08.

51-12.08 Principle-Based Design Guidance

51-12.08(01) Introduction

Roundabout design, due to the dynamic balancing of competing objectives and the effect that manipulation of geometric elements can have on roundabout performance tends to be iterative by its nature. Roundabout design can require numerous iterations to achieve the desired balance between geometric, operational, and safety factors. Though a design process is provided herein, the designer should understand that accordance with design principles and understanding of the inherent design tradeoffs are the central points of design regardless of the design procedure followed.
The FHWA Roundabout Guide foreword states the following:

Roundabout operation and safety performance are particularly sensitive to geometric design elements. Uncertainty regarding evaluation procedures can result in over-design and less safety. The ‘design problem’ is essentially one of determining a design that will accommodate the traffic demand while minimizing some combination of delay, crashes, and cost to all users, including motor vehicles, pedestrians, and bicyclists. Evaluation procedures are suggested, or information is provided, to quantify cost and how well a design achieves each of these aims. Since there is absolutely no optimum design, this guide is not intended as an inflexible ‘rule book,’ but rather attempts to explain some principles of good design and indicate potential tradeoffs. In this respect, the ‘design space’ consists of performance evaluation models and design principles such as those provided in this guide, combined with the expert heuristic knowledge of a design. Adherence to these principles still does not ensure good design, which remains the responsibility of the designer.

More so than for a conventional intersection or other design form, the geometric design of a roundabout intersection dictates its capacity and operational performance. The geometric and operational analyses, considered as distinct disciplinary pieces of project design and often performed separately for each project, are inseparable in roundabout design. Therefore, much of the information included herein invokes traffic-engineering terms and subject matter that centers on achieving operational goals while balancing them with safety and other considerations.

51-12.08(02) Roundabout-Design Process

As discussed previously, the general nature of the roundabout-design process is an iterative one. It is also a process in which minor adjustments in geometric attributes can have effects on the performance of the design. In the execution of this process, there must be an awareness of this iterative nature as well as an understanding that the designer may need to revert back to an earlier step for adjustment.

Due to the iterative process and the fact that the optimal position of the roundabout may not be finally determined until preliminary geometrics can be investigated, initial layout options should be prepared as rough concept drawings. This method allows an investigation of feasibility and compatibility of individual components before significant effort is invested in determining design elements.

Designing a roundabout can range from easy to complex. There is no easy process for the intersection design. A roundabout is not homogeneous in nature and cannot be standardized. There are many different types of roundabouts, such as single-lane, two-lanes, three-lanes, circles, ellipses, bypass lanes, snagged bypass lanes, double roundabouts, spirals, etc, in which a
number of combinations or multiple combinations of the above can appear in one roundabout. Each roundabout is unique, with each potential type applied in different situations in which site-specific problems require distinct solutions. The major differences in design techniques and complexity appear between a single-lane roundabout and a multi-lane roundabout where different principles apply.

Roundabout design is fundamentally holistic. The whole is more important than the parts. How the intersection functions as a single traffic-control device is more important than the actual values of the specific design components, e.g., a radius. However, how the parts interact with each other is also important. Although individual geometric values are not as important as the intersection operation as a whole, the values should be within ranges that succeed. Figure 51-12X provides an example of a holistic flowchart that guides a designer through the entire roundabout-design process.

51-12.08(03) General Design Steps

The following will most often apply. However, each roundabout requires a different design and thought process that is dependent on the unique design constraints, traffic volume, roadway speeds, topography, and alignments of the roadways. Not all aspects of design or the design process are included herein. However, the general design steps provided should be sufficient.

1. **Review of Existing Conditions.** Review the most recent site plans and roadway alignment information in an electronic format. Review existing roadways with respect to surrounding topography, centerlines, curb faces, edge of pavement, roadway lane markings, existing or proposed bicycle lanes, nearby crosswalks, environmental constraints, buildings, drainage structures, adjacent access points, multi-use paths, railroad crossings, school zones, and right-of-way constraints. This should include design constraints such as specific properties that may not be encroached upon or desired lane widths. Review available traffic studies, which should include projected design-year volume and assumptions for the proposed intersection or corridor project.

   These should provide adequate background regarding traffic conditions as well as the level of detail, design parameters, right-of-way constraints, and location for the proposed roundabout.

2. **Review of Future Conditions.** The future-intersection traffic operations and flows should be reviewed and discussed with the lead jurisdiction for the project. Possible issues including excessive delays should be considered in the design process and geometric criteria. Relevant future site plans, access points, and roadway cross-sections that can
affect the roundabout design should be provided, reviewed, and incorporated into the analysis and design.

Depending on the area, review the future projected morning, afternoon, off-peak, or mid-day peak-hour turning-movement volumes at the intersection. Use Figure 51-12Q, Traffic-Flow Worksheet, and a schematic diagram consisting of the future peak-hour turning-movement volumes at the intersection. To accurately identify the roundabout geometric and capacity needs, the information to be acquired prior to starting the capacity analysis or roundabout design is as follows:

a. future morning, afternoon, off-peak, or mid-day peak-hour turning-movement volumes, as deemed appropriate for the study area;
b. future percent trucks by approach for each peak hour;
c. design-vehicle type by turning movement, i.e., WB-50, WB-65, or special design vehicle;
d. vertical-alignment constraints;
e. right-of-way constraints;
f. base map, either aerial photo/mapping or topographic survey;
g. pedestrian volume, if significantly high; and
h. need for bicycle lanes or sidewalks.

3. Understanding of Specific Design Concerns. Prior to commencing a design, the designer must first understand the design concerns listed above and incorporate the needs into the roundabout design. After evaluating the traffic volumes, the designer should have an understanding of the number of lanes that will initially be required.

An approximate roundabout diameter can be determined based on the traffic needs, proximity to constraints, design vehicle, and the relative speeds of the roadways entering the intersection.

4. Performance of Capacity Analysis. After all of the pertinent information regarding the roadways, site, and traffic volumes have been obtained and an approximate roundabout diameter has been identified, a geometric analysis of the proposed roundabout should be performed using a roundabout capacity analysis software. See Figure 51-12Q, Traffic-Flow Worksheet, for assistance in inputting the traffic-volume data.

This will set the design requirements for the conceptual roundabout design. The morning and afternoon traffic volumes, or possibly a weekend peak depending on the study area, should be analyzed for the intersection. This will maximize the likelihood that the roundabout operates appropriately under all peak-hour traffic conditions. The final
results of this analysis will produce key information to include in the roundabout design, as follows:

a. initial estimated roundabout diameter;
b. entry-lane configurations at each approach;
c. future capacity for each approach;
d. minimum approach widths and entry radii;
e. delay in each approach and the overall delay for the intersection;
f. queue length for each approach; and
g. future level of service.

5. **Lane Configuration and Roundabout Placement.** Once the minimum design requirements have been established, a roundabout can be sketched by initially identifying the flow of traffic, lane configuration, approach-lane assignment requirements, the circulatory roadway width, and the exit lanes. This includes the placement of the roundabout's circle to roughly determine the lane configuration and location of the proposed roundabout. A skewed intersection angle or right-of-way constraints should be considered.

6. **Planning Initial Layout.** Once the capacity requirements have been identified, the initial conceptual layout should be refined further to satisfy the site’s specific design constraints. The concept should then be refined iteratively to develop a final concept drawing, without the use of exact values such as radii. Visual inspection of the concept can then further identify fastest path, right of way, deflection, leg angles, and other issues. The ability for the design vehicle to maneuver the roundabout should be checked. The roundabout geometry should be adjusted accordingly at this stage of the design process.

7. **Formalization of Roundabout Geometrics.** Once the general location and roundabout configuration have been preliminarily developed and all of the design issues have been resolved, a conceptual roundabout design can be completed. A roundabout design should be completed with respect to the face of curb for the intersection. For a multi-lane roundabout, the lane striping is as critical as the face-of-curb location to minimize entry- and exit-path overlap, provide proper lane widths and widening, and communicate the lane markings and possible spiral markings.

The horizontal geometry should be in accordance with relevant safety and capacity parameters. The design should incorporate the geometric roundabout parameters of entry-width, \( E \); average effective flare length, \( L' \); \( V \); entry angle, \( \varphi \); entry radius, \( R \); and inscribed-circular diameter, ICD. The values of \( E, L', \varphi, R, \) and ICD all directly relate to the capacity and safety of a modern roundabout.
8. **Design-Vehicle Check and Modifications.** Verify that the specific design vehicle is accommodated within the roundabout design. A software program such as AutoTurn should be used to verify proper truck turning radii through the roundabout for each movement. The truck-apron size, or width, should be identified. See Section 51-12.09(06) for assistance in sizing the truck apron. The information provided therein should be used for guidance purposes only, and should not be considered as a standard sizing chart. Each truck movement should provide a buffer space of 2 ft between the swept path of the truck and the face of curb.

9. **Safety and Fastest-Path Review.** Fastest-path design speed and other safety factors and design features such as φ should be checked. The specific fastest-path design should be developed and reviewed as adequate and reasonable, based on speed and deflection. If deficiencies or deviations in the design features and safety factors appear, the roundabout should be reanalyzed and redesigned either with small changes or by completely shifting alignments and geometry or the placement of the circle with an entire redesign effort, as an iterative process. See Section 51-12.09(03) for assistance in determining the fastest path.

10. **Accessorizing the Design.** Once a preliminary design with respect to the face of curb, and striping, if a multi-lane roundabout, has been completed, additional amenities should be considered at this stage. These include crosswalks, detached sidewalks, bicycle paths, and curb ramps.

At this stage of the design process, a form of approval or review consultation should be performed if desired. Once a roundabout has been properly designed with respect to horizontal geometry, other geometric and non-geometric design components must be completed for a roundabout to function as it was designed. These design components are key to the public driving the roundabout as it was intended without further safety or operational issues. These other elements include vertical profiles, signage, pavement marking, landscaping, lighting, and materials.

**51-12.08(04) Design Principles**

Provision should be made for an operationally-adequate facility with desirable safety performance. In the geometric design, these are often competing goals, as geometric elements that promote higher traffic flow often allow higher speeds into and through the roundabout. Issues relating to overall speed and speed consistency, between different traffic streams or between successive elements within the same traffic stream, are the most prevalent cause of safety problems.
The speed, capacity, and safety relationship should be in balance. The design process can require considerable iteration among geometric design, operational analysis, and safety evaluation. Minor adjustments in geometry can result in significant changes in safety or operational performance. Thus, the initial design will likely require revision and refinement to enhance the roundabout’s capacity and safety.

Since roundabout design is an iterative process, the initial concept drawings should be sketched. The individual components should be compatible with each other so that the roundabout will satisfy its overall performance objectives. Before the geometric details are finalized, the fundamental elements to be determined in the Scoping and Feasibility stage are as follows:

1. optimal size;
2. optimal position; and
3. optimal alignment and arrangement of the approach legs.

The following should also be incorporated into the roundabout design.

1. **Fastest-Speed Path.** This restricts operating speed by deflecting the paths of entering and circulating vehicles. See the FHWA *Roundabout Guide*, Chapter 6 and Exhibit 6-12, for additional information on vehicle-path curvature.

2. **Circulatory-Roadway Width.** This is the width between the outer edge of the inscribed diameter at the curb face of the roadway and the central-island-curb face. It is 1.0 to 1.2 times the widest entry width. It does not include the width of a traversable apron, which is defined to be part of the central island. The circulatory-roadway width defines the roadway width, curb face to curb face, for vehicle circulation around the central island.

3. **Exit Radius.** This is the radius of curvature of the outside curb face at the exit.

4. **Exit Width.** This defines the width of the exit where it meets the inscribed circle. It is measured perpendicularly from the right curb-face edge of the exit to the intersection point of the left curb-face edge and the inscribed circle.

**51-12.08(05) Design Composition**

Design composition consists of the selection and arrangement of geometric design elements and their relationships to one another. In composing a design, the tradeoffs of safety, capacity, and cost should be recognized and assessed throughout the design process. The effect of adding to one component of design often impacts another. Figure 51-12Y identifies such tradeoffs.
51-12.09 Geometric Design

See the FHWA Roundabout Guide, Chapter 6, for fundamental design principles as guidance. This Section provides guidelines and details for geometric design which do not appear in the Guide. This Section also provides information specific to a two-lane roundabout’s entries.

51-12.09(01) Design Speed

See the FHWA Roundabout Guide, Section 6.2.1.2.

51-12.09(02) Vehicle Path

Determine the smoothest, fastest path, using a spline, possible for a single passenger car, in the absence of other traffic, without regard to lane pavement markings, traversing through the entry, around the central island, and out the exit. The critical fastest path is most often the through movement, but can be a right-turn movement.

Use the FHWA Roundabout Guide, Exhibits 6-5 and 6-7, for a single-lane design with low pedestrian activity. Use Exhibit 6-5 to determine the radius values for the R1, R2, and R3 fastest-speed paths. Use Exhibit 6-7 to determine the radius value for R5 fastest-speed path. Do not use Exhibit 6-6, as the lane lines should not be considered in a multi-lane roundabout for fastest-speed analysis. See Section 51-12.09(03) for evaluating fastest-speed paths for a multilane roundabout. The R4 value does not control the fastest-speed path but should be checked to determine speed consistency. The vehicle-path offset of 5 ft as shown in Exhibits 6-5 and 6-7 is measured from the concrete curb face, and not the edge of the pavement line. If the approach to the roundabout has a centerline pavement marking on the left side and no curb face, the offset should be 3 ft from the centerline pavement marking. Figures 51-12Z and 51-12AA describe the vehicle-path radii. The entry-path curvature is measured on a curved path near the yield point over which the tightest radius occurs.

See Figure 51-12LL for determination of the entry-path curvature.

51-12.09(03) Creating a Fastest-Speed Path, or Spline Curve

1. Curb Offsets. Use the curb offsets shown in Figure 51-12CC. To determine the speed, the fastest path allowed by the geometry should be as shown in Figure 51-12CC. This is the smoothest, flattest path possible for a single vehicle, in the absence of other traffic
and without regard to lane markings, traversing through the entry, around the central island and out the exit.

2. **Draw the Spline Curve.**

   See Figure 51-12DD, Spline Curve Through Movement, for the locations of the points described below.

   a. Choose points A through C on the first 5-ft curb offset from the splitter island. Choose three points that are approximately 5 ft apart that will approximate the path of an approaching vehicle. Choose a point outside the 165-ft line, and another inside the 165-ft line.

   b. Choose point D on the 5-ft curb offset from the entry curve.

   c. Choose point E on the 5-ft curb offset from the central island.

   d. Choose point F on the 5-ft curb offset from exit curve.

   e. Choose points G through I, or G₁ through I₁, on the 5-ft offset from the right side exit curb. It can be appropriate to check the left side instead of the right side. The side is dependent on the anticipated driving path of the vehicle and the roadway alignment. Choose three points that are approximately 5 ft apart that will approximate the path of an exiting vehicle. Choose a point outside the 165-ft line, and another inside the 165-ft line.

   f. Establish a point just upstream from the start of the spline at point J. The beginning of the spline will be tangent to the 5-ft curb offset.

   g. Establish a point just downstream from the end of the spline at point K. The end of the spline will be tangent to the 5-ft curb offset.

3. **Modify the Spline Curve.** Check the spline created in item 2 above to determine if it violates the 5-ft curb offsets. Measure the distance between the face of curb and the spline curve at points A through I, or visually inspect whether the spline curve violates the curb offsets.

   The spline will likely slightly violate the 5-ft curb offset. Use engineering judgment to determine if the spline should be modified.
If the spline is between the curb offset and the curb or outside of the curb offset, as shown in Figure 51-12FF, it should be modified.

Evaluate the spline as a whole to determine if it appears to be a path that a vehicle will use. The beginning or end of the spline should likely be pulled farther away from the roundabout itself.

4. Measure R Values.
   a. Once an acceptable spline is created, fit arcs to the spline to measure the R values.
   b. Fit an arc onto the spline at a point that appears to be the tightest portion of the spline. This should occur prior to the yield line and not more than 165 ft from the yield line.
   c. Check the arc length. If the arc length is not 65 to 80 ft, recreate it to try to obtain an arc length within this range.
   d. Measure the radius of the arc.
   e. Repeat to find values for R1, R2, and R3.
   f. To find R4, measure the radius of the 5-ft curb offset from the central island.
   g. To find R5, create a spline that is tangent to the three curb offsets. These are the 5-ft splitter-island offset on the entry, the 5-ft offset on the inside of the right turn, and the 5-ft splitter-island offset on the exit that define the R5 path, as illustrated in Figure 51-12IF. Check that the arc does not cross the curb offsets.

51-12.09(04) Speed Consistency

In addition to achieving the appropriate design speed for the fastest movements, the relative difference in speeds between consecutive geometric elements should be minimized. The relative difference in the speeds between conflicting traffic streams should also be minimized. The maximum speed differential between movements should be not more than 12 mph as shown in the FHWA Roundabout Guide, Section 6.2.1.5. The R2 values for radius and speed are lower than the R1 values for a single-lane entry. However, this is seldom achievable for a multi-lane entry. For either a single- or multi-lane entry, the R2 values should be lower than the R3 values.
The R1 and R2 values should be used to control exit speed. For the through path, R1 should be greater than R2. R2 should be less than R3, but R3 should not be less than R1. For the left-turning path, R1 should be greater than R4.

The R1 to R4 relationship will most often be the most restrictive for speed differential at each entry. However, the R1, R2, and R3 relationship should also be reviewed, to ensure that the exit design is not overly restrictive in regard to speed. Design criteria in the past advocated relatively tight exit radii to minimize exit speeds. The current best practice suggests a more relaxed exit radius for improved drivability. Speeds at roundabout exits are still low due to R2 speeds and the short distance between R2 and the exit leg, rendering R3 practically irrelevant as a speed control.

For calculation of the exit speed at R3, NCHRP Report 572, Equation 5-4A should be used.

51-12.09(05) Design Vehicle

The standard design vehicle is the WB-65. Community-sensitive design considerations can suggest that larger or smaller vehicle accommodations are warranted. The appropriate jurisdiction should be consulted for selecting a design vehicle. Use of the facility by unconventional vehicles, e.g., farm vehicles, oversized loads, should be researched. The design should be modified accordingly so that such vehicles can be accommodated. The design vehicle can have an impact on the truck-apron width. The inscribed-circle diameter, the width of the circulatory roadway, and the central-island diameter are interdependent. Once two of these are established, the remaining measurement can be determined. However, the circulatory-roadway width, entry and exit widths, entry and exit radii, and entry and exit angles should also be considered in accommodating the design vehicle and providing deflection.

To ensure that smaller vehicles encounter sufficient entry deflection at a normal roundabout, a truck apron, or a raised low-profile area around the central island, is usually necessary. It should be capable of being mounted by a large truck’s trailer, but should be uncomfortable for a car or SUV to traverse. The roundabout should be designed such that select vehicles, usually a school or transit bus, will not require use of the truck apron. However, to keep entering and circulating speeds to a minimum, a vehicle larger than the select vehicle type may have to track onto the truck apron.

The width of the circulatory roadway should be determined from the width of the entries and the turning requirements of the proposed design vehicle. It should always be at least as wide as the maximum entry width, and can be up to 1.2 times the maximum entry width.

A multilane roundabout can be designed to accommodate large trucks. The most common method is to assume that a truck will use two lanes, by tracking into the adjacent lane, to enter,
circulate, and exit the roundabout. Alternatively, a roundabout can be designed so that a truck can remain in one lane as it traverses the intersection. This approach is less commonly used, since the roundabout must be larger, possibly resulting in increased right-of-way needs, higher cost, and a potential for increases in certain types of crashes. It can be applicable where truck volume represents a high percentage of the overall traffic.

As with a single-lane roundabout, the left- or U-turn movements will determine the width of the truck apron. In determining the apron width, a worst-case scenario should be assumed, where a truck’s cab and front tires stay completely within the inside circulatory lane. Vehicle turning-path templates or turning-path softwares can be used.

51-12.09(06) Considerations for Large Vehicles

Field observations have shown that most semi-trucks entering a multi-lane roundabout take up two lanes at the entry, therefore not allowing another vehicle to travel beside the truck on a two-lane circulatory roadway. Depending on the angle of entry and the size of the roundabout, a truck can travel completely in the outside lane with sufficient space for another vehicle to travel next to the truck. Where truck volume is high, it may be necessary to post a warning sign. No other vehicles should drive next to or pass a truck in a roundabout, unless the roundabout has been designed to specifically allow for trucks to travel side-by-side with another vehicle. Field observations have shown that where a car and truck enter a roundabout side-by-side, the smaller vehicle tends to accelerate ahead of the truck or slows to avoid driving beside the larger vehicle.

A secondary consideration associated with a large truck in a roundabout is the potential for overturning or shifting loads. There is no simple solution in relation to geometry to completely prevent load shifting or rollover. Load shifting or load shedding can lead to property damage, congestion, and delay. A vehicle whose load has shifted or has been shed is expensive to clear, especially if it occurs at a roundabout with high traffic volume. Where such problems persist, combinations of geometric features often exist, as follows:

1. long, straight high-speed approach;
2. inadequate entry deflection or too much entry deflection;
3. low circulating flow combined with excessive visibility to the right;
4. significant reduction of the turning radius on the circulatory roadway, due to spirals with arcs that are too short or elliptical geometries with too large a difference in the major and minor axes;
5. cross-slope changes on the circulatory roadway or the exit; and

6. outward sloping cross-slope on the inside lane of the circulatory roadway.

A problem for some vehicles can occur if speeds are low due to a combination of grades, geometry, sight distance, and driver responsiveness. An articulated large-load vehicle with a center of gravity at 8 ft above the ground can overturn in a 65-ft-radius curve at a speed as low as 15 mph. See TRL Report LR788.

A layout designed to mitigate the characteristics describe above will be less prone to load shifting or load shedding. Abrupt changes in the cross-slope should be avoided. Pavement-surface tolerances should be complied with.

**51-12.09(07) Non-Motorized Vehicles**

The splitter island’s desirable width from face of curb to face of curb is 8 ft, and the minimum width is 6 ft, within the pedestrian-refuge area. The desirable crosswalk width in the splitter island, from outside to outside of white edge lines, is 10 ft, and the minimum width is 7 ft. See the FHWA Roundabout Guide, Exhibit 6-26, for more information.

**51-12.09(08) Alignment of Approaches and Entries**

A key factor is deflection at entry, which is independent of roadway centerlines. Entry deflection should not be generated by sharply curving the approach road to the left close to the roundabout, then sharply to the right at entry.

The FHWA Roundabout Guide Exhibit 6-18 and accompanying text do not represent current policy for roundabout design. The centerline of roadway should not pass through the center of the inscribed circle. An offset should be provided in a multi-lane roundabout to the left of the center of the central island. An offset of 40 to 60 ft can be provided between opposing entries, or the distance shown in Figure 51-12JJ can be provided to achieve proper deflection and appropriate fastest-path R1 speeds.

**51-12.09(09) Entry Width**

Entry width is measured perpendicularly from the outside curb face to the inside curb face at the splitter-island point nearest to the inscribed circle.
A narrow entry tends to promote safety. However, a WB-65 design vehicle will require an 18- to 22-ft width entry path for a single-lane approach, depending on skew angle, to be able to make a right turn. A wide entry can cause confusion about whether the entry should be marked as multi-lane or single-lane. Increasing the effective flare length or entry width will increase capacity, or increasing both can produce a dramatic increase in capacity. Effective flare length of an approach can be as short as 15 ft or as long as 330 ft. Where the effective flare length exceeds 330 ft, its impact on capacity can become minimal. Therefore, a full approach should be added.

51-12.09(10) Circulatory-Roadway Width

The circulatory-roadway width need not remain constant. A variable circulatory-roadway width should be used where a multi-lane entry is appropriate for the major road, but only single lane approaches are necessary on the minor road.

51-12.09(11) Central Island

The central island is always a raised, non-traversable area encircled by the circulatory roadway. This area should also include a traversable truck apron, if necessary. The island should be raised and can be landscaped to enhance driver recognition of the roundabout upon approach and to limit the ability of the approaching driver to see through to the other side of the roundabout. The inability to see through the roundabout also reduces or can eliminate headlight glare at night and driver distraction caused by other vehicles in the circulatory roadway.

The center or highest portion of the central island’s ground-surface should be raised. The ground slope in the central island should not be steeper than 6:1. Concrete, stone, wood, or other material used to make a wall within the central island may be prohibited in certain speed zones. Use of such treatments should be discussed with the Office of Traffic Safety or the local municipality prior to design. Landscaping in the central island is further addressed in the FHWA Roundabout Guide, Section 7.5.

The outside 6 ft of the central island, excluding the truck apron, should be a low-cut grass surface or other low-maintenance surface to maintain visibility to the left upon entry, and forward and circulatory visibility within the circulatory roadway.

51-12.09(12) Inscribed-Circle Diameter

For the recommended inscribed-circle-diameter range, see Section 51-12.04(08), and Figures 51-12K and 51-12J. Also see the FHWA Roundabout Guide, Section 6.3.1.

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51-12.09(13) Splitter Island

The maximum overall height of a splitter island’s landscaping or hardscaping above the top of the curb should be approximately 1.5 ft. See the FHWA Roundabout Guide, Section 6.3.8.

See Figure 51-12KK for details for a typical splitter island.

51-12.09(14) Entry Radius, Right-Turn Bypass Lane, Path Overlap, and Deflection

The minimum entry radius should be as shown on Figure 51-12LL. Capacity will increase with increased entry radius, but entry speed will also increase.

Entry radius is not the same as R1. R1 should be greater than R2, and not less than R2 as stated in the FHWA Roundabout Guide.

A bypass lane allows vehicles to circumvent a roundabout, providing additional capacity. A bypass lane should be used only if other geometric layouts fail to provide acceptable traffic operations. The decision to use a bypass lane should consider pedestrian and right-of-way constraints. A bypass lane can provide significant benefits to a roundabout’s function. Provision for a large amount of right-turn demand can be made by means of a free-flow bypass lane, which allows vehicles to bypass the roundabout and merge into the exiting stream of traffic. It can also be made by means of a semi-bypass or partial bypass lane, which can or cannot include a vane island, which requires approaching vehicles to yield to traffic leaving the adjacent exit.

Choosing the proper alternative is determined from the volume of right turns and the available space. Capacity analysis with or without the right-turn flows will confirm the best choice. If there is high pedestrian volume, the use of a full-bypass lane should be avoided.

The FHWA Roundabout Guide’s Figures 6-42 and 6-43 illustrate two types of layouts for a bypass lane. The layout shown in Figure 6-43 is not recommended because right-turning drivers must look to the left at an angle of greater than 90 deg. The layout shown in Figure 6-42 is preferred for a full-bypass because the right-turning traffic has an exclusive exit without conflicts between other exiting traffic if the merge distance is sufficient where the auxiliary lane must be dropped downstream.

An alternative that can be superior to that shown in Guide Figure 6-43 is a partial right turn that still keeps the right-turning vehicle from making a through movement while preserving adequate sight to the left for circulating or exiting traffic. A vane island or pavement markings can be
used depending on space, alignment, entry angle, and the need to improve the vehicle-retention effect of the geometry. Figure 51-12P shows the addition of a partial or vehicle-retaining bypass lane at the upper leg of a roundabout. There are other features that can accompany this treatment. Two such features are shown in the figure. One is the narrowing of the downstream circulatory roadway by having the adjacent splitter island protrude into the circulatory roadway. The other is to pull the far right-hand curb on the north approach off the inscribed circle to aid the separation between entering or circulating traffic and right-turning traffic.

The addition of conflicts with multiple traffic streams entering, circulating, and exiting the roundabout in adjacent lanes, should be considered in designing a multi-lane roundabout.

The natural path of a vehicle is the path a driver will take based on the speed and orientation imposed by the roundabout’s geometry. While the fastest path assumes that a vehicle intentionally cuts across lane markings to maximize speed, the natural path assumes that there are other vehicles present, and all drivers will attempt to stay within their proper lanes. The natural path should be determined by assuming that a vehicle will stay within its lane to the yield point. At the yield point, the vehicle will maintain its natural trajectory into the circulatory roadway, continue into the circulatory roadway, and exit with no sudden changes in curvature or speed. Roundabout geometry that tends to lead a vehicle into the wrong lane can result in operational or safety deficiencies.

Path overlap occurs where the natural paths of vehicles in adjacent lanes overlap or cross one another. It most commonly occurs at an entry where geometry of the right-hand entry lane tends to lead vehicles into the left-hand circulatory lane. However, vehicle-path overlap can also occur at an exit where exit geometry or striping tends to lead vehicles from the left-hand circulatory lane into the right-hand exit lane. Figure 51-12MM illustrates an example of entry-path overlap at a multi-lane roundabout. The left-lane geometry directs the approach vehicle into the central island, and the right-lane geometry directs the approach vehicle toward the inside circulatory lane, creating entry-path overlap.

Figure 51-12NN provides a method for checking entry- and exit-path overlap at a multi-lane roundabout. To avoid path overlap, the desirable tangent length is 40 to 50 ft for the entry-path tangent and at least 40 ft for the exit-path tangent. The minimum tangent length to avoid entry- and exit-path overlap is 26 ft. Path overlap can be avoided if approximately 5 ft of space is provided between the face of the central-island curb and the extension of the face of curb on the splitter island.

Figure 51-12 OO shows the preferred method of avoiding path overlap. This method is consistent with the FHWA Roundabout Guide Exhibit 6-46, and is the preferred design for a multi-lane entry. An inner entry curve should first be drawn such that once the edge of the splitter-island curve is extended across the circulatory roadway, the line is tangent to the central
island as shown. Once the innermost-lane geometry has been designed, and it has been determined that there will be no path overlap, the adjacent lane can be designed. The radius of the smaller entry curve will vary depending upon the approach geometry and the fastest-speed path, but will range from 65 to 110 ft. A curve with a radius of greater than 150 ft, or a tangent section, is then fitted between the entry curve and the outside edge of the circulatory roadway.

Another method is to start with a larger, sweeping inner curve and then provide a smaller-radius curve near the approach that is tangent to the central island. This method is also described in the FHWA Roundabout Guide, Section 6.4.3.1.

The objective of this design technique is to place the entry curve at the optimal location so that the extension of the inside entry lane at the yield point forms a line tangent to the central island. This concept should be used for a multi-lane entry design, and is also recommended for a single-lane entry. Figure 51-12PP illustrates the result of proper entry design.

If the entry curve is located too close to the circulatory roadway, it can result in path overlap. If it is located too far away from the circulatory roadway, it can result in inadequate deflection, i.e., entry speed which is too fast. A multi-lane roundabout without path overlap can have adequate deflection to control entry speed. Improved path overlap can result in increased fastest-path speed. A technique for reducing the entry speed without creating path overlap is to increase the inscribed-circle diameter of the roundabout. The inscribed circle of a double-lane roundabout should be of at least 140 to 200 ft diameter (see Figure 51-12J), to achieve a satisfactory entry design. However, increasing the diameter will result in a slightly faster circulatory-roadway speed. The entry speed and circulatory-roadway speed should be in balance. This can require iteration of design, speed checks, and path-overlap checks.

The technique of offsetting the approach alignment to the left of the roundabout center is effective at increasing entry deflection (see Figure 51-12JJ). However, this also decreases the entry angle, which can create unsafe entry conditions, deficient line of sight, and unbalanced lane utilization if offset too far to the left. It also reduces the deflection of the exit on the same leg, which will increase the fastest-path speed at the entry. Therefore, the distance from the approach offset to the roundabout center should be kept to a minimum to maximize its effectiveness in design and safety for pedestrians. A typical offset is 20 to 30 ft from the center of the inscribed circle.
51-12.09(15) Lane-Drop Taper and Exit Design

At a multi-lane roundabout, it is common to drop one travel lane downstream of an exit. A vehicle leaving a roundabout is typically traveling between 15 and 25 mph and is accelerating away from the intersection. The taper should not start until after the exit radius is complete. There should also be a short parallel section before beginning the taper, if practical. An exit taper should be designed assuming a 35-mph design speed using the appropriate formula for a lane drop at lower than 40 mph. If a different design speed is determined to be applicable, contact the Production Management Division’s Roadway Standards Team.

The right, or outside, lane should be dropped. The likely traffic volume exiting the roundabout on each individual exit lane should be evaluated. If there is a substantially higher traffic volume that will be using the outside lane, it is beneficial to drop the left, or inside, lane instead.

The exit lane should be designed to promote a smooth natural drive path for a right-turning vehicle. The exit curve should start at the central island where the entry curve to the left ends, and extends past the pedestrian refuge to delineate the edge of the splitter island (see Figure 51-12KK). The lane will narrow from the circulating roadway width past the pedestrian refuge to match with the departing lane. The radius of the exit curve is larger than the entry curve to improve the ease of exit. A design that reduces the probability of a vehicle braking in the circulating lane or at the exit will minimize the likelihood of a crash at the exit. This larger radius does not translate into a faster speed where the exit speed is controlled by the circulating speed, R4, plus acceleration to the exit crosswalk.

Where a free-flow right-turn bypass lane is utilized, the design of the merge should consider the relative speeds of the two conflicting streams of traffic, and provide the necessary lengths for the parallel section and merge section.

51-12.09(16) Vertical-Alignment Considerations

See the FHWA Roundabout Guide, Section 6.3.11.

51-12.09(17) Clear Zone

Clear-zone guidance for a roundabout installation requires consideration of the approach speed, fastest-path speed, adjacent side slopes leading into and through the roundabout, and AADT. Guidance for determination of the clear-zone width is provided in Section 49-2.0 and the AASHTO Roadside Design Guide. The speed of a vehicle approaching an intersection and the
speed allowed through an intersection, along with the AADT and side slopes, will determine the required clear-zone width.

A stop-controlled intersection located in a high-speed rural area will require less clear-zone width than a traffic-signal-controlled intersection, as drivers are required to slow down to stop. As an approaching vehicle reduces speed, it can be appropriate and desirable to reduce the corresponding clear-zone width. The need for a clear zone and right-of-way acquisition should be balanced. The yield condition for a roundabout is similar to that for the stop-controlled intersection. The horizontal geometry of the roundabout requires a driver to slow down on the approach and through the roundabout.

The approaching speed transition distance is determined from the posted speed limit and the deceleration needed to enter the roundabout in accordance with the fastest-speed path calculation, R1 value. Section 51-12.09(18) and Figure 51-12QQ describe how to determine the roundabout’s approach layout for a high-speed highway.

The design speed used to determine the clear-zone width around the perimeter of the roundabout is the average of the entry speed, measured at R1, and the circulatory-roadway speed, measured at R2. The average fastest-path speed, \((R1 + R2)/2\), of 25 to 30 mph, will produce a clear-zone width of 7 to 18 ft, depending on AADT. The exit ramps of an interchange are also considered to be low speed in close proximity of the approach to the roundabout. In an urban area, lateral clearance is used rather than clear-zone width to determine the minimum distance to a fixed object such as a utility pole, fire hydrant, tree, etc. In a rural area, the clear-zone width is used, based on the design speed, AADT, and sideslopes.

The sideslopes adjacent to a roundabout are relatively flat to accommodate a small terrace and a multi-use path around the perimeter. If the multi-use path is not installed at the same time as the roundabout, the area can be graded so that a path can be installed in the future with minimal regrading. On an approach where vehicle speeds are 40 mph or lower, and on the perimeter of the roundabout beyond the multi-use path, the sideslopes should be 4:1 or flatter. They may be steeper if clear-zone requirements can be satisfied, or local impacts preclude the use of such gentle slopes.

Central-island clear-zone width is considered to be within a low-speed environment, and therefore should be in accordance with the lateral-clearance requirements for an urban street, 2 ft back from the face of curb. Certain landscaping materials and treatments which are dependent upon the approach speed should be considered in the central-island landscaping design. See the FHWA Roundabout Guide, Section 7.5, for additional guidance on central-island landscaping.
51-12.09(18) Approach Curbs

A low-speed approach requires vertical-face curbs of 6 in. height in the area of the splitter island on both sides of the roadway and on the splitter island. The purpose of the vertical-face curbs is to control the fastest-speed paths at the roundabout entrances and exits.

A high-speed approach is used with a rural cross section. A rural cross section for an undivided highway has shoulders without curbs on the outside. On a divided highway, the cross section has shoulders without curbs on both the inside and outside leading up to the roundabout. A high-speed approach requires a transition section to the roundabout, where the shoulders will narrow and vertical curbs will be introduced. See Figure 51-12QQ for an example of the high-speed approach layout. The figure shows the layout of the gore area for the beginning of the splitter island and the curb-and-gutter layout as the driver approaches the yield line. The painted gore area transitions into a raised concrete center curb type C followed by a sloping curb and gutter of 4 in. height for a short distance as shown. The curb transitions both horizontally and vertically as it approaches the roundabout. At the nose where the curb and gutter begins, the curb face is 4 to 6 ft from the driving lane, or has a shoulder of 4 to 6 ft width on the left side of the approach.

In a rural area, the painted gore and the curbs serve to alert a driver approaching a roundabout of changing roadway conditions, and that travel speed should be reduced. Driver awareness that conditions are changing is accomplished through the use of roadway curvature, channelization, lighting, landscaping, or signage. Total curb length starting from the yield line should be the deceleration distance required to reduce from the approach speed to the fastest-path design speed measured at R1.

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Example 51-12.1. The posted speed limit is 55 mph, and deceleration to approximately 25 mph produces a desirable total raised-curb length distance of approximately 400 ft for the splitter-island side of the roadway. Approximately 230 ft of the 400 ft should include a sloping curb of 4 in. height. The remaining 170 ft should include a vertical curb of 6 in. height. If the posted speed limit instead is 40 mph, deceleration to 25 mph will produce a desirable total raised-curb length of approximately 185 ft, all of which should be vertical curb of 6 in. height. Deceleration-distance is provided in the AASHTO Policy on Geometric Design of Highways and Streets, Exhibit 10-73. In using this exhibit, the approach speed limit should be used as the design speed. Differing approach conditions can produce different deceleration distances.

On the roundabout approach, the minimum length of vertical curb on the right side of the travelway should be the greater of 25 ft prior to the bicycle-path ramp or 100 ft prior to the yield line. The vertical-curb installation will enforce the fastest-speed path geometry. On the exit, the curb on the right-hand side also should be long enough to control exit speed and minimally
should be the greater of 25 ft past the bicycle-path ramp or 100 ft past the exit measured from the ICD.

Drainage should be considered in the area of the curb and gutter by providing curb turnouts or inlet structures.

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51-12.09(19) Sight Distance

Stopping sight distance should be provided so that a motorist can recognize the need to slow down and stop, if necessary. The FHWA Roundabout Guide, Section 6.3.9, should be followed in calculating stopping sight distances for the approaches, circulatory roadway, and crosswalks.

Regarding intersection sight distance, the FHWA Roundabout Guide, Section 6.3.10, provides one method for calculating required visibility to the left as a vehicle approaches a roundabout. However, the section recommends that an approaching driver should be able to see a considerable distance up the preceding approach, based on a conservative methodology for calculating sight distance, which can sometimes be undesirable.

An alternative method for calculating sight distance to the left requires that a driver approaching a roundabout should be able to see only to the yield line of the entry to the left once the driver approximately 50 ft ahead of the yield line. Either of these two methods is acceptable for calculating sight distance to the left.

Restricting visibility to the left until a driver is approximately 50 ft ahead of the yield line on an approach reduces the possibility of a crash. Restricting vision in this manner does not interfere with intersection stopping sight distance. There are benefits to making the central island more visible and reducing sight lines through the central island to the opposite side of the roundabout. See the FHWA Roundabout Guide, Section 7.5, for landscaping within the central island and approaches. Where practical, visibility should be restricted through the central island so that the roadway on the opposite side of the island will not be visible to a motorist approaching the roundabout.

Signs should be located such that they will not block clear-sight areas. However, chevron signs located in the central island will likely be located within a sight area.

In calculating the sight lines, adequate sight distance should be provided for snow of up to 1 ft depth in the central island. If the roundabout is within or near a horizontal curve, adequate sight distance should also be provided.
51-12.09(20) Landscaping Considerations

See the FHWA Roundabout Guide, Section 7.5.

51-12.09(21) Bicycle Provisions

The minimum bicycle-ramp width should be 4 ft between the roadway and the multi-use path, such that they angle 25 to 35 deg toward the path where the bicyclist exits the roadway, as shown in Figure 51-12W. Where a bicyclist re-enters the roadway, the ramp should likewise angle 25 to 35 deg toward the roadway. For applications pertinent to a multi-use path, see Section 51-7.0 and 51-12.07(03).

A perpendicular ramp should not be provided between the multi-use path and the roadway that will require a bicyclist to stop or nearly stop forward motion to enter one facility or the other. Each roundabout location should include a bicycle ramp between the roadway and a shared-use path. Where the shared-use path is not installed with the initial construction, the designer should determine whether or not the perimeter of the roundabout should be graded for future path installation.

51-12.10 Traffic-Control Design

51-12.10(01) Signage

The overall concept for roundabout signage is similar to that for other intersection types’ signage. Proper regulatory control, advance warning, and directional guidance are required to provide positive guidance to the roadway user. Signs should be located where roadway users can easily see them when they need the information in advance of the condition. Signs should not obscure pedestrians, motorcyclists, or bicyclists. Urban and rural applications require different signs or sign spacing. For the connecting highways, sign selection should be coordinated with the district Office of Traffic and the local public agency to maintain consistency on the facility.

The MUTCD governs the design and placement of signs. Also see the FHWA Standard Highway Signs manual and the INDOT Sign Design Guide for more information.

1. Regulatory Signs. The appropriate regulatory signs shown Figure 51-12RR are described below.
a. “Yield” Sign, R-2A. This should be placed both in the splitter island, and on the right side of each approach. If sight distance is limited, a “One Way” sign, R6-2R, should be placed under the splitter-island side’s “Yield” sign on each approach to establish the direction of traffic flow within the roundabout. A “To Traffic From Left” sign, may be placed under the right-side “Yield” sign on each approach to reinforce that the circulating traffic has the right of way.

b. “One Way” Sign, R6-1R. This should be placed in the central island opposite each entrance and mounted above the chevron sign to emphasize the direction of travel within the circulatory roadway. A single sign along with four chevrons is recommended.

c. “Keep Right” sign, R4-7. This should be placed at the nose of each raised-curb splitter island.

A lane-use sign, such as R3-8, should not be used for a single-lane entry. For a multi-lane entry, operational requirements will dictate where the R3-8 sign should be used.

The preferred R3-8 sign is modified to show a fishhook symbol to better identify roundabout-lane designations. This sign incorporates the placement of a dot under the left-pointing arrow, if present, which graphically depicts the presence of the central island. A dot under or beside the arrow should be used only for the left-most lane.

2. Warning Signs. The appropriate warning signs shown Figure 51-12SS are described below. The amount of warning a motorist needs is related to site-specific intersection conditions and the vehicular speeds on the approach roadways.

a. Circular Intersection Sign, W2-6. This should be placed on each approach in advance of the roundabout. An optional “Ahead” educational plate, W16-9p, should be placed below the W2-6 sign. Below the W16-9p plate, if placed, but below the W2-6 sign if not, an advisory-speed plate, W13-1, should be placed. The speed shown on the advisory-speed plate should not be higher than the design speed of the circulatory roadway. For closely spaced roundabouts, these signs may be omitted. See item 4 below for guidance as to where these signs may be omitted.

b. “Yield Ahead” sign, W3-2. This should be placed on each approach. For closely-spaced roundabouts, this sign may be omitted. See item 4 below for guidance as to where these signs should be omitted.
c. “Pedestrian Crossing” Sign. Use of this sign should be coordinated with the district Office of Traffic or the local public agency. If there is a school crossing at the roundabout, the school advance-warning sign assembly with arrow, S1-1 and W16-7p, is required at the crosswalk location, and in advance of the school crosswalk. If there is no school crossing at the roundabout, the pedestrian-crossing sign assembly, W11-2 and W16-7p, or the school-crossing sign assembly should be placed in front of the crosswalk on the approach and on the exit. A rural roundabout will not have pedestrian accommodations, and will not require pedestrian-sign assemblies. However, if pedestrians are anticipated, the pedestrian-sign assemblies described above are required. If the crosswalk is not considered to be part of the intersection because of its distance from the circulatory roadway, pedestrian-crossing accommodations and their design should be in accordance with the MUTCD and the Americans with Disabilities Act guidelines for a mid-block crossing.

Pedestrian-crossing signs should be located so as to not obstruct an approaching driver’s view of the “Yield” sign or pedestrians standing at the crosswalk.

d. Bicycle Sign Assembly, D11-1 and M7-4. This may be required to designate the exit to the bicycle path.

Flashing beacons may be used with some warning signs as a long-term awareness technique for an area with approach speeds of 45 mph or higher, or for an area with limited sight distance where an emphasis on advance warning is deemed necessary.

3. **Guide Signs.** These provide drivers with needed navigational information. They are particularly needed at roundabouts to reduce driver confusion. Overhead guide signs can be considered at a high-capacity multi-lane roundabout approach to guide motorists into the proper travel lane to navigate the roundabout properly and help avoid lane changing within the roundabout. The appropriate guide signs are described below.

a. **Intersection Destination and Direction Sign.** This should be placed at each approach in a rural location. It should be considered in an urban or suburban location where space allows, and its use is not determined to be inappropriate. The decision to install this sign is based on available right of way, agency preference, roadway speed, and type. This sign should be used only where necessary, and where it does not cause sign clutter. This sign may not be necessary at a local-street roundabout or in an urban setting where there are no significant destinations and the majority of users are familiar with the site.
The diagrammatic style is preferred over the text style, since the diagrammatic style reinforces the form and shape of the approaching intersection, and clarifies to the driver how to navigate the intersection. Examples of both styles are shown in Figure 51-12TT. If space is limited, or sign spacing becomes an issue, a text-style sign or overhead diagrammatic guide sign may be used.

b. Overhead Lane-Use Sign. This should be used at high-traffic-capacity location on a National Highway System route, or at an interchange location with multiple approach lanes. By providing destination guidance to the driver in advance, the vehicle will more likely be in the correct lane at the roundabout approach. The driver will be discouraged from making a lane change within the roundabout.

Qualifying criteria include two or more approach lanes, higher AADT, lane splits approaching the roundabout, dual turn lanes, a major route that turns instead of continuing straight through the roundabout, closely-spaced roundabouts, unfamiliarity of drivers, and documented crash patterns related to improper lane usage. All arrows should point up. Each individual sign is placed over each travel lane. See the urban roundabout layout example in Figure 51-12UU. The arrow is placed over the center of the lane. Sign placement should be coordinated with the district Office of Operations, the Production Management Division’s Traffic Review team, and the local public agency. If overhead lane-use signs are used on an approach, the diagrammatic-style sign is not required. The diagrammatic-style sign shows destinations and directions, but it does not depict proper lane assignments. See the MUTCD for the appropriate font style, vertical clearance, letter sizing, and other design and placement elements.

See the AASHTO Standard Specifications for Structural Support for Highway Signs, Luminaires and Traffic Signals for overhead-sign support design guidance.

c. Exit Guide Sign in Splitter Island. This reduces the potential for driver confusion. It is used to designate the destination of each exit from the roundabout. It is a conventional-intersection direction sign. An exit guide sign with a route-number sign should include the cardinal direction and arrow. The arrow should point to the right at 45 deg. For a freeway ramp, the route continuation should be shown on the exit guide sign, as shown in Figure 51-12VV.

d. Route Confirmation Sign. For an intersection of one or more numbered routes, a route-sign assembly should be placed after the roundabout exit to reassure drivers that they have selected the correct exit at the roundabout. A confirmation assembly should be placed not more than 500 ft beyond the intersection. The
assembly should be placed close enough to the intersection so that it can be seen by a driver in the circulatory roadway.

A junction assembly consisting of a “Jct”, M2-1, auxiliary sign should be considered with the appropriate route-number sign in advance of the roundabout. See the MUTCD for additional guidance.

4. **Urban Signage Considerations.** An urban intersection tends to exhibit lower speeds. Consequently, fewer and smaller signs can be placed than in a rural setting. However, some indication of street names in the form of exit guide signs or street-name signs should be included. The proposed signage layout should be reviewed to ensure that sign clutter will not reduce its effectiveness. Sign clutter can be avoided by prioritizing signage and eliminating or relocating lower-priority signs. A sample signage plan for an urban application is shown in the FHWA *Roundabout Guide*, Exhibit 7-15.

5. **Rural and Suburban Signage Considerations.** Route guidance emphasizes destinations and numbered routes rather than street names. The exit guide sign should be visible but discrete from within the roundabout. A sample signage plan for a rural application is shown in the FHWA *Roundabout Guide*, Exhibit 7-16.

6. **Closely-Spaced Roundabouts.** Roundabouts can be installed less than 1/8 mile apart. This situation can cause signage challenges due to longitudinal space constraints between the roundabouts. As a result, some signs may be eliminated between the roundabouts. Visibility distance is based on stopping sight distance. The roundabout warning assembly signs W2-6, W2-6p, and W13-1, and “Yield Ahead”, W3-2, may be eliminated between roundabouts if the visibility distance between the roundabouts exceeds the minimum visibility distance shown in Figure 51-12WW. Other signs may be eliminated after review with the district Office of Operations and the local public agency. The warning assembly signs and “Yield Ahead” sign should be placed at the approaches to the first roundabout in the series.

7. **Roundabout in Close Proximity to Railroad Crossing.** This can present signage challenges due to safety concerns and the installation of additional signs where a number of signs are already required. Since each railroad-crossing situation is unique, the Production Management Division’s Railroads Team, the district Office of Operations, and the local public agency should be contacted for approval of the signage and marking layout if the railroad crossing is 750 ft or less from the roundabout.

8. **Short-Term Awareness Techniques.** Once a roundabout is first installed, there can be a need to raise driver awareness of geometric features or signs. There can be a need to mitigate a certain situation, such as a driver failing to yield on a certain approach, after a
roundabout has been in operation. The district traffic engineer should be contacted for guidance. Traffic-control devices are not expected to accomplish what the geometric design cannot. Available mitigation measures to increase driver awareness include providing portable changeable-message signs or installing orange flags on top of the “Yield” signs during the first six months of operation.

9. **Illuminated Bollard.** An illuminated bollard, as shown in Figure 51-12XX, has proven beneficial to roundabout safety. Its use has thus far been limited as it is not shown in the MUTCD as a traffic-control device in the form shown in Figure 51-12XX. It is installed at the nose of the splitter island, or the end of the island farthest away from the circulatory road, aiding drivers during periods of low visibility. The use of an illuminated bollard which displays a traffic-control message is considered experimental and requires the necessary approvals for each project. An illuminated bollard without a sign displayed on it is permitted.

### 51-12.10(02) Pavement Markings

Pavement markings for a single-lane roundabout are discussed in FHWA Roundabout Guide, Section 7.2, and the MUTCD.

The FHWA Roundabout Guide and the MUTCD do not address pavement markings for a multi-lane roundabout, therefore, guidance is provided below. Figures 51-12E and 51-12F show the terminology that is used for such pavement markings.

1. **Markings for a Multi-Lane Roundabout.** The objective of using such markings is to provide direction to motorists so that they can traverse a roundabout without changing lanes. This improves safety and traffic operations, and educates drivers about proper lane use. They can be critical to the successful functioning of a roundabout. This is true where unusual peak hour turning patterns, i.e., double right turn, double left turn, occur. Pavement markings can also be used as a low-cost option to retrofit an older roundabout or traffic circle with problematic geometry.

Pavement-marking principles include the following.

a. Turning patterns or volumes should be accommodated without inconsistencies. The marking scheme should accommodate all of the individual movements without requiring drivers to change lanes inside the roundabout during different peak hours. This can be accomplished by tracking each movement through the intersection for both peak hours. If conflicts within the same peak hour or
between different peak hours cannot be resolved after trying different pavement-marking schemes, the designer should consider either of the following:

(1) partial spiral markings that accommodate the major traffic streams as long as they do not create conflicts; or
(2) markings not placed in the circulatory roadway.

b. Lane-use control arrows should designate and reinforce correct lane usage. The lane-designation pavement markings should match the lane designation signs used at the approaches. The fishhook sign style and pavement marking should be used.

c. Lane changing within the circulatory roadway should be discouraged. Discouragement is achieved by directing drivers into the correct lane before they reach the yield line and maintaining lane consistency throughout the intersection. Providing relatively long approach lanes facilitates this objective by allowing drivers more time to get into the correct lane as they approach the yield line. Lane-use control arrows can be repeated on the approaches, with two or three sets of arrows being desirable.

d. Approach arrows should be oriented relative to and should define the exit road that can be accessed from each specific approach lane, and should be consistent with lane-use signs. Approach arrows or stripes should be consistent with circulatory-roadway arrows, stripes, and signs. Unbalanced lane use, i.e., most vehicles using one lane instead of balancing out evenly on two or three lanes, should be discouraged by means of the selection of proper lane designations which achieve the most-even distribution of traffic possible.

e. For a flared approach, where one or more lanes are added, lane stripes should extend back from the yield line as far as reasonably possible.

f. Line types should convey the correct message. A 2:1 ratio of 4-in.-width line to gap should be used with a 12-ft line and 6-ft gap for lane lines on the approaches, circulatory roadway, and exits. The approach and circulatory-roadway marking may be a solid white line instead to discourage lane-change behavior.

g. The yield line should be broken white, with a width of 12 to 24 in., with 2 ft of line and 2 ft of gap.

h. Pavement markings and signs should be an integral part of the geometric design and should be developed concurrently with a concept’s design.
i. Concentric-circles markings should not be used in the circulatory road. These markings cause indecision, lane-use imbalance, decreased capacity, and the potential for exit crashes.

j. Markings at closely-spaced roundabouts should function as one integrated system. This provides guidance for drivers to select the lane they must use for their ultimate destination before entering the system and to be able to traverse multiple intersections without changing lanes. Extra lanes that are not needed solely for capacity purposes may be added for lane continuity.

k. Guide dots should be used to direct motorists from the yield line into the proper circulating lane. A dotted white line of 6 in. width should be used, with a 1-ft line and a 10- to 15-ft gap.

l. The triangles markings or the word “Yield” placed at each approach can be integrated into the pavement-marking layout to enhance the safety and visibility of the yield on entry. These should be placed perpendicular to the lane at the yield line.

2. **NCUTCD Guidelines.** The National Committee on Uniform Traffic Control Devices (NCUTCD) has approved draft guidelines regarding pavement markings at a roundabout. While this information has not yet been formally adopted for use, it can be used if it does not conflict with the principles defined above or in the FHWA *Roundabout Guide*. Otherwise, there is no pavement-marking guidance regarding a multi-lane roundabout. The circulatory-road markings of a combination of a solid line and a short dashed line shown in the NCUTCD draft should not be used. Instead, a 12-ft line with a 6-ft gap should be used for the circulatory road. Adoption of the NCUTCD draft is subject to FHWA approval prior to implementation. The designer should verify with INDOT or the local public agency prior to implementation.

3. **Spiral Hatching.** Where one lane of the circulatory road is to be spiraled out away from the central island, a spiral hatching near the central island should be used. However, drivers often ignore these markings, drive over them, and do not transition into the desired lane. Therefore, an irregularly-shaped central island with the curbed island at the same location should be placed instead of the spiral hatching. If an irregularly-shaped central island is used, the design vehicle should still be able to circulate adjacent to the central island without overrunning the innermost curb.

4. **Curb Faces.** These may be painted with reflective paint. This is used to aid drivers in identifying curb locations at night.
51-12.10(03) Lighting

1. **Introduction.** A driver should be able to perceive the general layout and operation of an intersection in time to make appropriate maneuvers. If a facility is designed for use by a high volume of motor vehicles, pedestrians and bicycles, or mopeds, it should be illuminated. Additional illumination guidance is available in Section 502-4.0, and the IESNA Publication DG-19-08, *Design Guide for Roundabout Lighting.*

2. **Need for Illumination.** The need varies depending on the location of the roundabout.

   a. **Urban Area.** An urban roundabout should be illuminated if all or most of its approaches are illuminated as necessary to improve the visibility of pedestrians and bicyclists, or if the facility is intended for use as a transition speed zone.

   b. **Suburban Area.** Illumination should be considered for safe traffic flow if the conditions are present as follows:

      (1) one or more approaches is illuminated;

      (2) competing non-roadway illumination in the vicinity can distract a driver's attention, e.g., parking lot, car-dealership lot, or filling station;

      (3) high nighttime traffic volume is anticipated; or

      (4) pedestrian traffic is anticipated, or approaches have sidewalks.

      A continuity of illumination level should be provided between the approaches and the roundabout to avoid distracting drivers and to minimize the need for drivers’ eyes to adjust to changing lighting levels.

   c. **Rural Area.** A rural roundabout should be illuminated. Retroreflective pavement markings and signs should be placed whether or not illumination is provided.

3. **General Recommendations.** Illumination enables drivers to see and navigate the geometric features of the roundabout at night. Lighting also facilitates mutual visibility among the users.

   Illumination should be provided on the approach nose of each splitter island, at all conflict areas where traffic is entering the circulating stream, and where the traffic streams separate to exit the roundabout.
The roundabout should be illuminated from the outside in toward the center to improve the visibility of the central island and the visibility of circulating vehicles to vehicles approaching the roundabout. Illumination from the central island outward should not be used, since vehicles become shadows against the light, and thus, less visible. If it is desired to illuminate specific objects in the central island, ground-level lighting should be used within the central island that shines upward toward objects and away from the nearest roadway. Accent lighting and roadway lighting should be placed on separate electrical disconnects for the purpose of blackout protection.

Pedestrian crossings should be illuminated. Illumination of bicycle merging areas should be considered.

4. **Light-Pole Placement.** The placement of a light pole relative to the curb should be determined from the speed environment in which the roundabout is located and the potential speed of an errant vehicle that can be expected. The lateral placement of a light pole should be in accordance with the AASHTO *Roadside Design Guide* requirements for clearance, clear zone, or obstruction-free zone.

The crosswalks should be illuminated so that pedestrians are in positive contrast. A light pole should be placed 10 to 30 ft ahead of the crosswalk. A pole should be offset 10 ft from the roadway to allow adequate spacing for large vehicles such as trucks and farm equipment to safely maneuver the roundabout.

Where pedestrian facilities do not exist, a layout that assumes a future multi-use path should be considered. The layout should consider longitudinal light-pole placement as described above, and lateral offset to avoid major facility relocations where pedestrian or bicycle paths are provided in the future.

Light supports or other poles or hazards should not be placed within a splitter island, or on the right-hand perimeter immediately downstream of an exit point.

**51-12.10(04) Work-Zone Traffic Control**

Traffic maintenance can be accomplished by means of partial-width construction or intersection closure.

If partial-width construction is used, traffic should be routed through the roundabout in a counterclockwise direction to train drivers as to the proper direction of travel at the intersection, especially during the final stages of construction prior to opening the intersection.
51-12.11 Public Involvement

51-12.11(01) Introduction

Although it is not always possible to achieve, the goal regarding public involvement should be to build consensus and support for the road improvements under consideration. This Section provides information about educating the public and obtaining public input regarding a roundabout. Figure 51-12ZZ includes example materials that can be used in the public-involvement process. See the Public Involvement Manual for more information. The designer should coordinate with the local public agency.

51-12.11(02) Educating the Public

Public acceptance of a roundabout is required where it appears to be an acceptable technical solution. Misconceptions still exist due to unfamiliarity and failure to distinguish a roundabout from an old-style traffic circle. Therefore, public involvement and roundabout education should be the first steps in leading the public toward acceptance of a roundabout. Public resistance has been common prior to construction of a roundabout. Once the roundabout has been constructed and is operating, public opinion has most often been favorable. Due to public misunderstandings about roundabouts, a public-involvement campaign should include roundabout education for elected officials, staff members, and the general public. These types of public-involvement processes allow local governments to partner with INDOT.

In educating the public, topics for consideration are as follows:

1. basic roundabout concepts and terminology;
2. differentiation between a modern roundabout and a traffic circle (see Figure 51-12YY);
3. data showing the increased safety benefits of a roundabout in comparison to other intersection types;
4. existing accepted locations within the State;
5. use at a high-speed intersection;
6. cost and right-of-way impacts compared to other options;
7. older or inexperienced drivers;
8. how to maneuver a roundabout, or the understanding of how a roundabout operates;
9. efficiency compared to other intersection types;
10. trucks;
11. snow removal;
12. proximate drives;
13. safe accommodation of pedestrians while in accordance with ADA requirements; and
14. bicycles.
51-12.11(03) Public-Involvement Techniques

These are similar to those utilized for another type of road project. Input should be sought after initial investigations have been conducted, and a roundabout has been determined feasible by INDOT, but before design commences.

Before design, consideration of public input should consider the degree to which public education has occurred. Public-involvement processes should be adapted to each individual project based on coordination with local officials, the project manager, and the Public Information Division.

A public-information meeting can be used to educate the public about a roundabout and to hear their feedback. Such a meeting provides the opportunity to dispel roundabout myths, and it permits the public to be involved in the planning process and have its questions and concerns addressed. Informational materials at a public meeting can include the following:

1. exhibits mounted on foamcore boards;
2. brochures;
3. video photography from before and after;
4. still photography from before and after;
5. project-specific materials; or
6. public-comment forms.

Websites can also be used to provide the public with roundabout information.
SIDEWALK PLAN

SIDEWALK CLEAR WIDTH

NOTES:

1. The minimum width of a sidewalk adjacent to a buffer is 5 ft. The minimum width of a sidewalk adjacent the curb is 6 ft. Where the sidewalk width is less than 5 ft a passing space is required every 200 ft. The passing space minimum clear dimensions are 5 ft by 5 ft. See Figure 51-1B, Sidewalk Passing Space.

2. The sidewalk grade may meet but not exceed the grade of the adjacent roadway.

Figure 51-1A
SIDEWALK WIDTH <5 ft for MORE THAN 200 ft

NOTES:

1. The minimum width of a sidewalk adjacent to a buffer is 5 ft. The minimum width of a sidewalk adjacent the curb is 6 ft. Where the sidewalk width is less than 5 ft a passing space is required every 200 ft. The passing space minimum clear dimensions are 5 ft by 5 ft.

2. The sidewalk grade may meet but not exceed the grade of the adjacent roadway.

SIDEWALK PASSING SPACE

Figure 51-1B
CURB RAMP COMPONENTS, DESIGN ELEMENTS, AND DESIGN CRITERIA

Figure 51-1C
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<table>
<thead>
<tr>
<th>Component</th>
<th>Perpendicular Curb Ramp</th>
<th>Parallel Curb Ramp</th>
<th>Blended Transition Curb Ramp</th>
<th>Depressed Corner Curb Ramp</th>
<th>Diagonal Curb Ramp</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ramp</td>
<td>Required</td>
<td>Required</td>
<td>n/a</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>Blended Transition</td>
<td>n/a</td>
<td>n/a</td>
<td>Required</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Turning Space</td>
<td>Top of Ramp (1)</td>
<td>Bottom of Ramp</td>
<td>n/a</td>
<td>Bottom of Ramp</td>
<td>Top of Ramp</td>
</tr>
<tr>
<td>Clear Space</td>
<td>Required</td>
<td>Required (2)</td>
<td>Required</td>
<td>Required (2)</td>
<td>Required</td>
</tr>
<tr>
<td>Detectable Warning</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
</tr>
<tr>
<td>Surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flared Side</td>
<td>Required (3)</td>
<td>n/a</td>
<td>Required (3)</td>
<td>Required (3)</td>
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</tr>
<tr>
<td>Return Curb</td>
<td>Required (4)</td>
<td>n/a</td>
<td>Required (4)</td>
<td>Required (4)</td>
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<tr>
<td>Crosswalk Markings</td>
<td>Preferred</td>
<td>Preferred</td>
<td>Required</td>
<td>Preferred</td>
<td>Required</td>
</tr>
</tbody>
</table>

(1) A turning space is not required for a one-way directional curb ramp where there is no change of direction at the top of the ramp.
(2) The clear space is coincident with turning space.
(3) A flared side is required where the curb ramp is adjacent a walkable surface and acceptable where curb ramp is adjacent a non-walkable surface.
(4) A return curb is acceptable where the curb ramp is adjacent a non-walkable surface or walkable surface with street furniture or other features that make the area non-walkable. Not acceptable where curb ramp is adjacent a walkable surface.

**CURB RAMP COMPONENTS, DESIGN ELEMENTS, AND DESIGN CRITERIA**

Figure 51-1C
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<table>
<thead>
<tr>
<th>Component</th>
<th>Design Element</th>
<th>Design Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ramp</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>4 ft min</td>
</tr>
<tr>
<td></td>
<td>Running Slope</td>
<td>8.00% preferred (8.33% max)</td>
</tr>
<tr>
<td></td>
<td>Cross Slope</td>
<td>1.50% preferred (2.00% max) (1)</td>
</tr>
<tr>
<td></td>
<td>Length</td>
<td>15 ft max</td>
</tr>
<tr>
<td><strong>Blended Transition</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Width</td>
<td>4 ft min</td>
</tr>
<tr>
<td></td>
<td>Running Slope</td>
<td>1.50% preferred (2.00% max) (2)</td>
</tr>
<tr>
<td></td>
<td>Cross Slope</td>
<td>1.50% preferred (2.00% max) (1)</td>
</tr>
<tr>
<td></td>
<td>Clear Dimension</td>
<td>4 ft x 4 ft min (3)</td>
</tr>
<tr>
<td><strong>Turning Space</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Running Slope</td>
<td>1.50% preferred (2.00% max)</td>
</tr>
<tr>
<td></td>
<td>Cross Slope</td>
<td>1.50% preferred (2.00% max) (1)</td>
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<tr>
<td></td>
<td>Clear Dimension</td>
<td>4 ft x 4 ft (4) (5)</td>
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<tr>
<td></td>
<td>Counter Slope</td>
<td>5.00% max</td>
</tr>
<tr>
<td><strong>Detectable Warning Surface</strong></td>
<td></td>
<td>Width: full ramp, blended transition, or turning space width (7)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Depth: 2 ft in direction of pedestrian travel</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Location: back of curb (8) (9)</td>
</tr>
<tr>
<td></td>
<td>Flared Side</td>
<td>Slope: 10.00% max</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Location: (10)</td>
</tr>
<tr>
<td></td>
<td>Return Curb</td>
<td>Location: (11)</td>
</tr>
</tbody>
</table>

(1) The cross slope may exceed 2.00% where it is acceptable for the pedestrian street crossing cross slope to exceed 2.00% in accordance with Section 51-1.05.
(2) Where a 4 ft sidewalk is provided at the back of the blended transition the maximum allowable running slope is 5.00% (4.50% preferred).
(3) Where the turning space is constrained at the back of sidewalk the minimum clear dimension is 4 ft x 5 ft. The 5-ft dimension is in the direction of the ramp running slope.
(4) The clear space is provided within the pedestrian street crossing and wholly outside the parallel vehicle travel lane for perpendicular curb ramps, blended transition curb ramps and diagonal curb ramps.
(5) The clear space is coincident with the turning space for parallel curb ramps and depressed corner curb ramps.
(6) Where the algebraic difference between the ramp running slope and counter slope is greater than 11.00%, a 2 ft minimum level strip should be provided at the bottom of the ramp. See Figure 51-1E, Change of Grade.
(7) The detectable warning surface is placed the entire width of the ramp, blended transition, or turning space.
(8) The detectable warning surface location and orientation depends on the distance from the bottom ramp grade break to the back of curb.
(9) Where the median width is less than 6 ft, a detectable warning surface should not be placed.
(10) A flared side is required where a curb ramp is adjacent a walkable surface. A flared side is acceptable where a curb ramp is adjacent a non-walkable surface.
(11) A return curb is acceptable where the curb ramp is adjacent a non-walkable surface or walkable surface with street furniture or other features that make the area non-walkable. Not acceptable where curb ramp is adjacent a walkable surface.

**CURB RAMP COMPONENTS, DESIGN ELEMENTS, AND DESIGN CRITERIA**

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<table>
<thead>
<tr>
<th>Alteration</th>
<th>Maintenance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open-Graded Surface Course</td>
<td>Crack Sealing and Filling</td>
</tr>
<tr>
<td>Mill and Fill, Mill and Overlay</td>
<td>Surface Sealing</td>
</tr>
<tr>
<td>Hot-in-Place Recycling</td>
<td>Chip Seal</td>
</tr>
<tr>
<td>Microsurfacing, Thin-Lift Overlay</td>
<td>Slurry Seal</td>
</tr>
<tr>
<td>Addition of New Layer of Asphalt</td>
<td>Fog Seal</td>
</tr>
<tr>
<td>Asphalt and Concrete Rehabilitation and Reconstruction</td>
<td>Scrub Seal</td>
</tr>
<tr>
<td>New Construction</td>
<td>Joint-Crack Seal</td>
</tr>
<tr>
<td></td>
<td>Joint Repair</td>
</tr>
<tr>
<td></td>
<td>Dowel Bar Retrofit</td>
</tr>
<tr>
<td></td>
<td>Spot High – Friction Treatment</td>
</tr>
<tr>
<td></td>
<td>Diamond Grinding</td>
</tr>
<tr>
<td></td>
<td>Pavement Patch</td>
</tr>
</tbody>
</table>

**ALTERATIONS VS. MAINTENANCE ACTIVITIES**

*Figure 51-1D*
CROSS SLOPE AT PEDESTRIAN STREET CROSSING

Figure 51-1E
The algebraic difference between a curb ramp slope and the gutter or pavement slope must not exceed 13.33%. A 2-ft wide level strip is recommended when the rate of grade change exceeds 11%. See the INDOT Standard Drawings.

$$\Delta G = S - CS$$

Where:

$\Delta G$ = Rate of Grade Change
$S$ = Running Slope of Curb Ramp or Sidewalk, % (Positive)
$CS$ = Counter Slope of Gutter or Sidewalk, % (Negative)

$$8.33\% - (-5\%) = 13.33\% > 11\%$$

Provide curb as required, may be monolithic with level strip.

Provide 2'-0 level strip if algebraic difference exceeds 11%
TIERED PERPENDICULAR CURB RAMP

ADJACENT WALKABLE SURFACE

Sidewalk
Sidewalk
Sidewalk
Sidewalk

Crosswalk Markings

Building or Other Constraint

Sidewalk
Sidewalk

Ramp

Flared Side
Flared Side

Curb

Crosswalk Markings

LEGEND

TS Turning Space

Detectable Warning Surface

CS Clear Space

PERPENDICULAR CURB RAMP

Figure 51-G
MIDBLOCK CROSSING CURB RAMP

PARALLEL CURB RAMP

Figure 51-1H
Where the median width is less than 6 ft, detectable warning surfaces should not be placed.

**Note:**

**CUT-THROUGH**

**PERPENDICULAR CURB RAMP IN-LINE OR OFFSET**

**MEDIAN PEDESTRIAN CROSSINGS**

*Figure 51-1 I*
NOTE:

1. A 4 ft sidewalk is required at the back of the blended transition where the running slope exceeds 2%.

CURB RAMP WITH 4 ft SIDEWALK BEHIND BLENDED TRANSITION

BLENDED TRANSITION CURB RAMP

Figure 51-1J
NOTE: Turning space is not required at the top of the ramp for a one-way directional perpendicular curb ramp.

LEGEND

Detectable Warning Surface

ONE-WAY DIRECTIONAL PERPENDICULAR CURB RAMPS

Figure 51-1K
DEPRESSED CORNER CURB RAMP

Figure 51-1L
NOTE:
1. A diagonal curb ramp should not be specified for new construction.

DIAGONAL CURB RAMP

Figure 51-1M
<table>
<thead>
<tr>
<th>Total No. of Marked or Metered Parking Spaces on Block Perimeter</th>
<th>Minimum Required Number of Accessible Parking Spaces</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 through 25</td>
<td>1</td>
</tr>
<tr>
<td>26 through 50</td>
<td>2</td>
</tr>
<tr>
<td>51 through 75</td>
<td>3</td>
</tr>
<tr>
<td>76 through 100</td>
<td>4</td>
</tr>
<tr>
<td>101 through 125</td>
<td>5</td>
</tr>
<tr>
<td>151 through 200</td>
<td>6</td>
</tr>
<tr>
<td>201 and over</td>
<td>4 % of total</td>
</tr>
</tbody>
</table>

Notes:
1. The IMUTCD contains provisions for marking on-street parking spaces
2. Metered parking includes parking metered by parking pay stations.
3. Where parking on part of the block perimeter is altered, the minimum number of accessible parking spaces required is based on the total number of marked or metered parking spaces on the block perimeter.

**ON-STREET PARKING**

**MINIMUM NUMBER OF ACCESSIBLE SPACES**

Figure 51-1N
PARALLEL PARKING ADJACENT WIDE SIDEWALK

PERPENDICULAR OR ANGLED PARKING

ACCESSIBLE PARKING

Figure 51-1 O
LEGEND:

- Detectable Warning Surface
- Ramp
- Pedestrian Pushbutton Assembly
- Wheelchair

1 The minimum required clear dimensions of a pushbutton clear space are 4 ft by 4 ft.

PUSHBUTTON CLEAR SPACE

Figure 51-1P
A pushbutton assembly should be centered adjacent a pedestrian clear space and centered relative to the crosswalk. Overlapping a pushbutton clear space with a curb ramp turning space is preferred.

A pushbutton assembly should not be placed more than 5 ft outside the crosswalk. A pushbutton assembly may only be placed adjacent a ramp with a running slope of 2% or less.

**LEGEND:**
- Detectable Warning Surface
- Ramp
- Pedestrian Pushbutton Assembly
- TS Turning Space/Pushbutton Clear Space

**PEDESTRIAN PUSHBUTTON ASSEMBLY OUTSIDE THE BACK EDGE OF SIDEWALK, PREFERRED**

Figure 51-1Q
1. The distance from a pushbutton assembly to face of the curb or edge of pavement should be between 1.5 ft and 6 ft and should not be greater than 10 ft. A minimum offset of 1.5 ft from the face of curb or edge of pavement will allow a wheelchair user to remain out of traffic while actuating the pushbutton assembly. A minimum offset of 1.5 ft also provides an appurtenances-free zone along the roadway.

2. A pedestrian pushbutton assembly should be adjacent a pedestrian clear space. Overlapping the pushbutton clear space with a curb ramp turning space is preferred.

3. A pushbutton assembly should not be placed more than 5 ft outside the crosswalk. A pushbutton assembly may only be placed adjacent a ramp with a running slope of 2% or less.

**LEGEND:**
- Detectable Warning Surface
- Ramp
- Pedestrian Pushbutton Assembly
- TS Turning Space\Pushbutton Clear Space

**PEDESTRIAN PUSHBUTTON ASSEMBLY WITHIN A SIDEWALK OR BUFFER**

Figure 51-1R
PEDESTRIAN PUSHBUTTON ASSEMBLY MOUNTING
HEIGHT AND SIDE REACH

Figure 51-1S
1. The face of a pedestrian pushbutton assembly must be aligned parallel to the direction of pedestrian travel on the associated crosswalk.

LEGEND:
- Detectable Warning Surface
- Ramp
- Pedestrian Pushbutton Assembly
- TS Turning Space

ORIENTATION OF PEDESTRIAN PUSHBUTTON ASSEMBLY

Figure 51-1T
<table>
<thead>
<tr>
<th>Design Element Factor</th>
<th>Cars</th>
<th>Cars/Trailers</th>
<th>Trucks</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mainline Traffic Data</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 Year ADT (A)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 Year ADT, Directional (B)</td>
<td>A x 0.60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DHV, Directional (DHV)</td>
<td>B x 0.135 (1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Traffic Composition</strong></td>
<td>(20-year projected)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars (D1)</td>
<td>(D1) Cars</td>
<td>C1=DHV x D1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars/Trailers (D2)</td>
<td>(D2) Cars/Trailers</td>
<td>C2=DHV x D2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trucks (D3)</td>
<td>(D3) Trucks</td>
<td>C3=DHV x D3</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Vehicles Per Hour @ Rest Area (VPH)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars Stopping (E1)</td>
<td>(E1) Cars</td>
<td>VPH1=E1 x C1</td>
<td>VPH2=E2 x C2</td>
<td>VPH3=E3 x C3</td>
</tr>
<tr>
<td>Normal Routes</td>
<td>.09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tourist Routes</td>
<td>.13</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Information &amp; Welcome Centers</td>
<td>.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars/Trailers (E2)</td>
<td>(E2) Cars/Trailers</td>
<td>VPH1=E1 x C1</td>
<td>VPH2=E2 x C2</td>
<td>VPH3=E3 x C3</td>
</tr>
<tr>
<td>Normal Stopping</td>
<td>.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trucks (E3)</td>
<td>(E3) Trucks</td>
<td>VPH1=E1 x C1</td>
<td>VPH2=E2 x C2</td>
<td>VPH3=E3 x C3</td>
</tr>
<tr>
<td>Normal Stopping</td>
<td>.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Parking Spaces</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars (T1) – Average Stop</td>
<td>(T1) Cars</td>
<td>P1=VPH1 x T1</td>
<td>P2=VPH2 x T2</td>
<td>P3=VPH3 x T3</td>
</tr>
<tr>
<td>at info. centers</td>
<td>.25 to .33 hr.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cars/Trailers (T2)</td>
<td>.33 to .50 hr.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trucks (T3) (2)</td>
<td>.50 hr.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Rest Room Requirements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Persons/Hour (PH)</td>
<td>VPH x 3.0 occupancy x .75 use</td>
<td>PH x 0.5</td>
<td>PH x 0.5</td>
<td></td>
</tr>
<tr>
<td>Number of Comfort Facilities – Men’s Room (M)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Comfort Facilities – Women’s Room (W)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) Assume 13.5% or the 20-year projected DHV, whichever is greater.
(2) Maximum of 80 truck and recreational vehicle parking spaces.

**DESIGN GUIDE FOR FREEWAY REST-AREA FACILITIES**

**Figure 51-2A**
Legend: 
“A” = Angle of Parking
“B” = Entrance Roadway Width
“C” = Exit Roadway Width
“D” = Parking Width
“E” = Total Width

DETAILS FOR PARKING SPACE

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>30</td>
<td>30</td>
<td>50</td>
<td>110</td>
</tr>
<tr>
<td>45°(1)</td>
<td>40</td>
<td>36</td>
<td>70</td>
<td>145</td>
</tr>
<tr>
<td>60</td>
<td>50</td>
<td>46</td>
<td>85</td>
<td>180</td>
</tr>
</tbody>
</table>

(1) Preferred angle design.

DESIGNS FOR ANGLE PARKING
(Based on WB-20 Design Vehicle)
Figure 51-2B
<table>
<thead>
<tr>
<th>Persons/Hour Using Rest Room During Design Hours (1)</th>
<th>Number of Facilities – Men’s Room (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Urinals (3)</td>
</tr>
<tr>
<td>0-105</td>
<td>2</td>
</tr>
<tr>
<td>106-225</td>
<td>4</td>
</tr>
<tr>
<td>226-315</td>
<td>6</td>
</tr>
<tr>
<td>316-375</td>
<td>8</td>
</tr>
<tr>
<td>376-435</td>
<td>10</td>
</tr>
<tr>
<td>436-500</td>
<td>12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Persons/Hour Using Rest Room During Design Hours (1)</th>
<th>Number of Facilities – Women’s Room (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Toilets (3)</td>
</tr>
<tr>
<td>0-105</td>
<td>6</td>
</tr>
<tr>
<td>106-225</td>
<td>10</td>
</tr>
<tr>
<td>226-315</td>
<td>14</td>
</tr>
<tr>
<td>316-375</td>
<td>18</td>
</tr>
<tr>
<td>376-435</td>
<td>20</td>
</tr>
<tr>
<td>436-500</td>
<td>24</td>
</tr>
</tbody>
</table>

**Notes:**

(1) See Figure 51-2A to determine the number of persons/hours.

(2) Dual men’s/women’s facilities (minimum of 2 each) should be provided. The number of fixtures should be divided equally among the rest rooms.

(3) At least one fixture should be handicapped accessible in each rest room provided. For additional criteria, see ADA Guidelines.

**GUIDELINES FOR COMFORT FACILITIES**

Figure 51-2C
Parking Layout Dimensions (ft) for 9 ft x 18 ft Stall of Various Length

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Diagram Location</th>
<th>Parking Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stall width, parallel to aisle</td>
<td>A</td>
<td>12.5 10.5 9.2 9</td>
</tr>
<tr>
<td>Stall length of line</td>
<td>B</td>
<td>27.6 23.6 21.0 18</td>
</tr>
<tr>
<td>Stall depth to wall</td>
<td>C</td>
<td>19.4 20.3 20.0 18</td>
</tr>
<tr>
<td>Aisle width between stall lines</td>
<td>D</td>
<td>12.1 12.8 23.0 19</td>
</tr>
<tr>
<td>Stall depth, interior</td>
<td>E</td>
<td>16.4 18.0 19.0 18</td>
</tr>
<tr>
<td>Module, wall to interior</td>
<td>F</td>
<td>47.9 55.1 62.0 63</td>
</tr>
<tr>
<td>Module, interior</td>
<td>G</td>
<td>44.9 53.1 61.0 63</td>
</tr>
<tr>
<td>Module, interior to curb face</td>
<td>H</td>
<td>45.9 52.5 59.4 60</td>
</tr>
<tr>
<td>Bumper overhang (typical)</td>
<td>I</td>
<td>2.0 2.3 2.5 2.5</td>
</tr>
<tr>
<td>Offset</td>
<td>J</td>
<td>6.6 2.5 0.7 0</td>
</tr>
<tr>
<td>Setback</td>
<td>K</td>
<td>13.1 9.2 4.9 0</td>
</tr>
<tr>
<td>Cross aisle, one-way</td>
<td>L</td>
<td>14.1 14.1 14.1 14.1</td>
</tr>
<tr>
<td>Cross aisle, two-way</td>
<td>---</td>
<td>24.0 24.0 24.0 24</td>
</tr>
</tbody>
</table>

NOTES:
1. See Section 51-1.0 for criteria on the number and dimensions of parking spaces for physically-challenged individuals.
2. If a parking-lot section is designated for subcompact vehicles, the stalls may be 8 ft x 15 ft for a 90° parking angle.
3. Stalls should be wider for commercial-vehicle parking.
4. Bumper overhang should be considered in placing lighting, railings, etc. Therefore, these appurtenances should be placed beyond dimension I in the diagram.
5. Only two-way traffic should be used with a 90° parking angle.

PARKING STALL DIMENSIONS

Figure 51-4A
SHALLOW SAWTOOTH PARKING

RECOMMENDED LENGTHS FOR BUS-LOADING AREAS
(Parking-and-Ride Lots)

Figure 51-4B
END OF CURB RETURN PROPERTY LINE, CROSSWALK, OR STOP LINE

CURB

FAR-SIDE BUS STOP

CURB

MID-BLOCK BUS STOP

END OF CURB RETURN PROPERTY LINE, CROSSWALK, OR STOP LINE

CURB

NEAR-SIDE BUS STOP

ON-STREET BUS STOPS

Figure 51-5A
NEAR-SIDE CORNER LOCATION

FAR-SIDE CORNER LOCATION

MID-BLOCK LOCATION

BUS TURNOUT DESIGNS

Figure 51-5B
RECREATIONAL ROAD NETWORK

Figure 51-6A
<table>
<thead>
<tr>
<th>Design Controls</th>
<th>Manual Section</th>
<th>Area Road</th>
<th>Circulation Road</th>
<th>Primary Access Road</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design-Year Traffic (Current AADT)</td>
<td>40-2.0</td>
<td>&lt; 100</td>
<td>≥ 100</td>
<td>≥ 100</td>
</tr>
<tr>
<td>Design Forecast Year</td>
<td>40-2.0</td>
<td>Current</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design Speed (mph)</td>
<td>40-3.0</td>
<td>10-20</td>
<td>10-20</td>
<td>25-35</td>
</tr>
<tr>
<td>Access Control</td>
<td>40-5.0</td>
<td>None (2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Level of Service</td>
<td>40-2.0</td>
<td>Desirable: B; Minimum: D</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cross-Section Elements</th>
<th>Manual Section</th>
<th>Travel Lane Width</th>
<th>Shoulder Width</th>
<th>Cross Slopes</th>
<th>Auxiliary Lane Lane Width</th>
<th>Shoulder Width</th>
<th>Obstruction-Free Zone (5)</th>
<th>Side Slopes Cut</th>
<th>New or Reconstructed Bridge Structural Capacity</th>
<th>Existing Bridge to Remain in Place Structural Capacity</th>
<th>Vertical Clearance (Recreational Road Under) New or Replaced Overpassing Bridge</th>
<th>Vertical Clearance (Recreational Road Over Railroad) (6) Ch. 69</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Width (4)</td>
<td>12 to 14 ft</td>
<td>2 to 4 ft</td>
<td>3 ft</td>
<td>10 or 11 ft</td>
<td>2 to 4 ft</td>
<td>2 ft</td>
<td>51-6.02(05), 45-1.0</td>
<td>51-6.02(05), 45-3.0</td>
<td>44-4.0</td>
<td>Ch. 69</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Typical Surface Type</td>
<td>HMA / Aggregate</td>
<td>Aggregate / Earth</td>
<td>6% if Aggregate; 8% if Earth</td>
<td>Desirable: 10 ft</td>
<td>Desirable: 2 ft; Minimum: 1 ft</td>
<td>Desirable: 3 ft</td>
<td>Desirable: 4.1; Maximum: 1½:1</td>
<td>Minimum: 0 ft (V-Ditch)</td>
<td>15 ft</td>
<td>14.5 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>HMA</td>
<td>Aggregate / Earth</td>
<td></td>
<td>Desirable: 10 ft</td>
<td>Desirable: 2 ft; Minimum: 1 ft</td>
<td>Desirable: 6.5 ft</td>
<td>Desirable: 4.1; Maximum: 1½:1</td>
<td>Minimum: 0 ft (V-Ditch)</td>
<td>23 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**GEOMETRIC DESIGN CRITERIA FOR RECREATIONAL ROAD**

*Figure 51-6B*
<table>
<thead>
<tr>
<th>Alignment Elements</th>
<th>Design Element</th>
<th>Manual Section</th>
<th>15 mph</th>
<th>20 mph</th>
<th>25 mph</th>
<th>30 mph</th>
<th>40 mph</th>
<th>50 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stopping Sight Distance</td>
<td>2-Lane (1b)</td>
<td>51-6.02(02), 42-1.0</td>
<td>80 ft</td>
<td>115 ft</td>
<td>155 ft</td>
<td>200 ft</td>
<td>250 ft</td>
<td>305 ft</td>
</tr>
<tr>
<td></td>
<td>1-Lane (1a)</td>
<td>160 ft</td>
<td>230 ft</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Passing Sight Distance</td>
<td>42-3.0</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>1090 ft</td>
<td>1470 ft</td>
<td>1835 ft</td>
<td>n/a</td>
</tr>
<tr>
<td>Intersection Sight Distance</td>
<td>46-10.0</td>
<td>170 ft</td>
<td>225 ft</td>
<td>280 ft</td>
<td>335 ft</td>
<td>445 ft</td>
<td>555 ft</td>
<td>n/a</td>
</tr>
<tr>
<td>Minimum Radius (e=4%)</td>
<td>51-6.02(04), 43-2.0</td>
<td>70 ft</td>
<td>125 ft</td>
<td>205 ft</td>
<td>300 ft</td>
<td>565 ft</td>
<td>930 ft</td>
<td>n/a</td>
</tr>
<tr>
<td>Superelevation Rate</td>
<td>51-6.02(04), 43-3.0</td>
<td>n/a</td>
<td>e&lt;sub&gt;max&lt;/sub&gt; = 4%</td>
<td>n/a</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal Sight Distance</td>
<td>51-6.02(04), 43-4.0</td>
<td>(7)</td>
<td>n/a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Curvature (K-value)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Crest</td>
<td>2-Lane (1b)</td>
<td>44-3.0</td>
<td>3</td>
<td>7</td>
<td>12</td>
<td>9</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-Lane (1a)</td>
<td></td>
<td>12</td>
<td>25</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td></td>
<td>Sag</td>
<td>2-Lane (1b)</td>
<td></td>
<td>10</td>
<td>17</td>
<td>26</td>
<td>37</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-Lane (1a)</td>
<td></td>
<td>27</td>
<td>44</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Maximum Grade</td>
<td>Level</td>
<td>44-1.02</td>
<td>8%</td>
<td>8%</td>
<td>7%</td>
<td>7%</td>
<td>7%</td>
<td>7%</td>
</tr>
<tr>
<td></td>
<td>Rolling</td>
<td></td>
<td>12%</td>
<td>11%</td>
<td>10%</td>
<td>10%</td>
<td>9%</td>
<td>8.5%</td>
</tr>
<tr>
<td>Minimum Grade</td>
<td>44-1.03</td>
<td>Desirable: 0.5%; Minimum: 0.0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**GEOMETRIC DESIGN CRITERIA FOR RECREATIONAL ROAD**

*Figure 51-6B (Continued)*
1. **1 Lane or 2 Lanes.** For Section 51-6.0 only, the following will apply:
   a. The criteria for one lane refer to two-directional traffic on a one-lane road.
   b. The criteria for two lanes refer to a two-lane roadway or a one-way roadway with either one or two lanes.

2. **Access Control.** Access to private individuals is not provided within the recreational area. However, access may be provided on the primary access road.

3. **Travel-Lane Width.** A total roadway width greater than 14 ft is not recommended for a one-lane road. For a one-lane road, the travel lane width is predicated upon the type of vehicle expected to use the facility.

4. **Shoulder Width.** Where a barrier is used, the graded width of shoulder should desirably be increased by 2 ft.

5. **Obstruction-Free Zone.** The minimum obstruction-free zone will be the shoulder width.

6. **Vertical Clearance (Recreational Road Over Railroad).** See Chapter Sixty-nine for additional information on railroad clearance under a highway.

7. **Horizontal Sight Distance.** For a given design speed, the necessary middle ordinate will be determined by the minimum radius and the stopping sight distance which applies at the site.
<table>
<thead>
<tr>
<th>User Type</th>
<th>Average Width (ft)</th>
<th>Average Length (ft)</th>
<th>Average Eye Height (ft)</th>
<th>Average Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bicycle</td>
<td>2.0</td>
<td>5.5</td>
<td>5.15</td>
<td>11</td>
</tr>
<tr>
<td>Bicycle with Trailer</td>
<td>3.7</td>
<td>9.5</td>
<td>5.25</td>
<td>11</td>
</tr>
<tr>
<td>Hand Cycle</td>
<td>2.1</td>
<td>6.0</td>
<td>3.15</td>
<td>9</td>
</tr>
<tr>
<td>Inline Skates</td>
<td>1.7</td>
<td>1.3</td>
<td>5.51</td>
<td>10</td>
</tr>
<tr>
<td>Kick Scooter</td>
<td>1.3</td>
<td>2.2</td>
<td>4.82</td>
<td>8</td>
</tr>
<tr>
<td>Manual Wheelchair</td>
<td>2.0</td>
<td>3.3</td>
<td>3.97</td>
<td>4</td>
</tr>
<tr>
<td>Power Scooter</td>
<td>1.9</td>
<td>3.7</td>
<td>4.33</td>
<td>6</td>
</tr>
<tr>
<td>Power Wheelchair</td>
<td>2.1</td>
<td>4.0</td>
<td>4.07</td>
<td>6</td>
</tr>
<tr>
<td>Power wheelchair &amp; dog</td>
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<td>3.9</td>
<td>3.84</td>
<td>4</td>
</tr>
<tr>
<td>Recumbent Bicycle</td>
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<td>6.2</td>
<td>4.13</td>
<td>14</td>
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<td>Segway5</td>
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<td>1.8</td>
<td>6.17</td>
<td>9</td>
</tr>
<tr>
<td>Skateboard</td>
<td>0.8</td>
<td>2.5</td>
<td>5.09</td>
<td>8</td>
</tr>
<tr>
<td>Stroller</td>
<td>1.7</td>
<td>4.1</td>
<td>4.36</td>
<td>3</td>
</tr>
</tbody>
</table>

**USER-TYPE DIMENSIONS AND SPEEDS**

*Figure 51-7A*
Bicyclist Operating Space

Figure 51-7B
SHARED-USE-PATH SEPARATION FROM ROADWAY WITH NO CURB

Figure 51-7C
<table>
<thead>
<tr>
<th>Roadway Speed Limit (mph)</th>
<th>Separation, $b^*$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 45$</td>
<td>20, desirable</td>
</tr>
<tr>
<td></td>
<td>10, minimum</td>
</tr>
<tr>
<td>$\geq 50$</td>
<td>24 to 35</td>
</tr>
</tbody>
</table>

* or roadway clear-zone width, whichever is greater

**SHARED-USE-PATH SEPARATION WIDTH FROM ROADWAY WITH NO CURB**

**Figure 51-7D**
SHARED-USE-PATH SEPARATION FROM ROADWAY WITH CURB

Figure 51-7E
<table>
<thead>
<tr>
<th>Roadway Speed Limit (mph)</th>
<th>Separation, $b^*$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 30$</td>
<td>5, minimum</td>
</tr>
<tr>
<td></td>
<td>3, minimum if parking permitted</td>
</tr>
<tr>
<td>35 or 40</td>
<td>5, minimum</td>
</tr>
<tr>
<td>$\geq 45$</td>
<td>10, minimum</td>
</tr>
</tbody>
</table>

* or roadway clear-zone width, whichever is greater

**SHARED-USE-PATH SEPARATION WIDTH FROM ROADWAY WITH CURB**

Figure 51-7F
<table>
<thead>
<tr>
<th>Bicycle and Pedestrian Traffic Composition</th>
<th>Recommended Pavement Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bicycle use with low-volume pedestrian use</td>
<td>10 ft</td>
</tr>
<tr>
<td>Bicycle use with high-volume pedestrian use, segment length ≤ 300 ft where right-of-way is restricted</td>
<td>10 ft</td>
</tr>
<tr>
<td>Bicycle use where pedestrian use is likely to be infrequent</td>
<td>8 ft</td>
</tr>
<tr>
<td>Bicycle use with low-volume pedestrian use, segment length ≤ 300 ft where right-of-way is restricted</td>
<td>8 ft</td>
</tr>
<tr>
<td>Bicycle use with high-volume pedestrian use</td>
<td>12 ft</td>
</tr>
<tr>
<td>High-volume bicycle use with low-volume pedestrian use</td>
<td>12 ft</td>
</tr>
<tr>
<td>High-volume bicycle and pedestrian use</td>
<td>≥ 14 ft</td>
</tr>
<tr>
<td>Path segment where queuing occurs, such as a road crossing</td>
<td>≥ 14 ft</td>
</tr>
</tbody>
</table>

PATH PAVEMENT WIDTH BASED ON PATH-USE TRAVEL COMPOSITION

Figure 51-7G
<table>
<thead>
<tr>
<th>Grade, $G$ (%)</th>
<th>Maximum Length of Segment (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$5 \leq G &lt; 7$</td>
<td>800</td>
</tr>
<tr>
<td>$7 \leq G &lt; 8$</td>
<td>400</td>
</tr>
<tr>
<td>$8 \leq G &lt; 9$</td>
<td>300</td>
</tr>
<tr>
<td>$\geq 9$</td>
<td>200</td>
</tr>
</tbody>
</table>

GRADE RESTRICTION
FOR PAVED SHARED-USE PATH

Figure 51-7H
<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Radius (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>36</td>
</tr>
<tr>
<td>15</td>
<td>56</td>
</tr>
<tr>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>25</td>
<td>156</td>
</tr>
<tr>
<td>30</td>
<td>260 *</td>
</tr>
</tbody>
</table>

*radius is based on a lean angle of 20 deg

DESIRABLE MINIMUM RADIUS OF HORIZONTAL CURVATURE FOR PAVED SHARED-USE PATH BASED ON LEAN ANGLE OF 15 DEG

Figure 51-7 I
Stopping Sight Distance, S, measured between cyclists along this line.

Angle is expressed in degrees

\[ M = R \left[ 1 - \cos \left( \frac{28.65 \times S}{R} \right) \right] \]

\[ S = \frac{R}{28.65} \left[ \cos^{-1} \left( \frac{R - M}{R} \right) \right] \]

Formula applies only if \( S \leq \) length of curve.

Line of sight is 28 in. above centerline of inside lane at point of obstruction.

\( R = \) Radius to centerline of inside lane, ft.
\( M = \) Distance from centerline of inside lane to obstruction, ft.

LATERAL CLEARANCE AT HORIZONTAL CURVE

FIGURE 51-7J
<table>
<thead>
<tr>
<th>$R$ (ft)</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>160</th>
<th>180</th>
<th>200</th>
<th>220</th>
<th>240</th>
<th>260</th>
<th>280</th>
<th>300</th>
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</thead>
<tbody>
<tr>
<td>25</td>
<td>2.0</td>
<td>7.6</td>
<td>15.9</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>50</td>
<td>1.0</td>
<td>3.9</td>
<td>8.7</td>
<td>15.2</td>
<td>23.0</td>
<td>31.9</td>
<td>41.5</td>
<td>--</td>
<td>--</td>
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<td>--</td>
<td>--</td>
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</tr>
<tr>
<td>95</td>
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<td>8.3</td>
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<td>15.5</td>
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<td>53.1</td>
<td>60.5</td>
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<td>1.0</td>
<td>2.2</td>
<td>4.0</td>
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<td>34.9</td>
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<td>7.0</td>
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<td>14.2</td>
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<td>8.2</td>
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<td>21.5</td>
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<td>28.5</td>
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<td>4.9</td>
<td>6.4</td>
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<td>12.1</td>
<td>14.3</td>
<td>16.8</td>
<td>19.5</td>
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<td>4.3</td>
<td>5.7</td>
<td>7.2</td>
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<td>8.6</td>
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<td>0.6</td>
<td>1.0</td>
<td>1.6</td>
<td>2.2</td>
<td>3.1</td>
<td>4.0</td>
<td>5.1</td>
<td>6.2</td>
<td>7.6</td>
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<td>10.5</td>
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<td>14.0</td>
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<td>900</td>
<td>--</td>
<td>0.2</td>
<td>0.5</td>
<td>0.9</td>
<td>1.4</td>
<td>2.0</td>
<td>2.7</td>
<td>3.6</td>
<td>4.5</td>
<td>5.6</td>
<td>6.7</td>
<td>8.0</td>
<td>9.4</td>
<td>10.9</td>
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<td>1000</td>
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<td>2.4</td>
<td>3.2</td>
<td>4.0</td>
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<td>6.0</td>
<td>7.2</td>
<td>8.4</td>
<td>9.8</td>
<td>11.2</td>
</tr>
</tbody>
</table>

**MINIMUM LATERAL CLEARANCE, $M$ (ft),
FOR HORIZONTAL CURVE

Figure 51-7 K
MINIMUM STOPPING SIGHT DISTANCE VS. GRADE BASED ON DESIGN SPEED

Figure 51-7L

If $S > L$, $L = 2S - \frac{900}{A}$

If $S < L$, $L = \frac{AS^2}{900}$

where $A = \text{Algebraic Grade Difference}\%$.

The shaded area represents $S = L$.

Height of cyclist’s eye = 4.5 ft. Height of object = 0 ft.

**MINIMUM LENGTH OF CREST VERTICAL CURVE, $L$, BASED ON STOPPING SIGHT DISTANCE**

Figure 51-7M
<table>
<thead>
<tr>
<th>Design Speed, mph</th>
<th>Stopping Sight Distance, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0% Grade</td>
</tr>
<tr>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td>12</td>
<td>63</td>
</tr>
<tr>
<td>15</td>
<td>85</td>
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<td>20</td>
<td>127</td>
</tr>
<tr>
<td>25</td>
<td>175</td>
</tr>
<tr>
<td>30</td>
<td>230</td>
</tr>
</tbody>
</table>

**STOPPING SIGHT DISTANCE FOR DOWNGRADE**

*Figure 51-7N*
<table>
<thead>
<tr>
<th>Speed Limit</th>
<th>Roadway Type</th>
<th>ADT</th>
<th>Proposed Treatments Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 30 mph</td>
<td>2 Lanes</td>
<td>&lt;12,000</td>
<td>1 or 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td>3 Lanes</td>
<td>&lt;12,000</td>
<td>1 or 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes with Raised Median</td>
<td>&lt;12,000</td>
<td>1 or 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12,000 ≤ ADT &lt; 15,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥15,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes without Raised Median</td>
<td>&lt;9,000</td>
<td>1 or 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9,000 ≤ ADT &lt; 12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td>35 mph or 40 mph</td>
<td>2 Lanes</td>
<td>&lt;12,000</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td>3 Lanes</td>
<td>&lt;9,000</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9,000 ≤ ADT &lt; 15,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥15,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes with Raised Median</td>
<td>&lt;9,000</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9,000 ≤ ADT &lt; 15,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥15,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes without Raised Median</td>
<td>&lt;12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td>≥ 45 mph</td>
<td>2 Lanes</td>
<td>&lt;12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td>3 Lanes</td>
<td>&lt;12,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes with Raised Median</td>
<td>&lt;15,000</td>
<td>2 + (3 or 4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥15,000</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>≥ 4 Lanes without Raised Median</td>
<td>&lt;12,000</td>
<td>[2 + (3 or 4)] or 5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≥12,000</td>
<td>5</td>
</tr>
</tbody>
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**RECOMMENDED TREATMENT OF SHARED USE PATH AND ROADWAY INTERSECTION**

Figure 51-7 O (Pg. 1 of 2)
**Level 1**  **Basic Crosswalk Treatment**
Standard crosswalk (two transverse lines)

**Level 2**  **Enhanced Crosswalk Treatment**
1) Longitudinal crosswalk markings (“Piano Key” or “Continental” pattern)
2) Raised midblock crosswalk (crossing elevated to match top of curb across entire width and length of crosswalk, formed with concrete or HMA, a plan detail is required.) This treatment may be considered only for local roadway functional classifications with a design speed of 25 mph or less.
3) For local projects, other high visibility crosswalk marking patterns such as diagonal crosswalk markings (“Zebra” pattern) may be used or textured pavement crosswalks with white retroreflective markings.

**Level 3**  **Refuge Islands and Bulbouts**
1) Median refuge islands
2) Split pedestrian crossover (SPXO – median refuge island with longitudinal offset between crosswalks)
3) Intersections bulbouts*
4) Midblock bulbouts*

*A bulbout is an extension of the sidewalk/curb area at a pedestrian or shared use path crossing and is designed to reduce the crossing length. A plan detail is required.

**Level 4**  **Flashing Beacons and Flashing LED Signs**
1) Ground-mounted flashing beacons
2) Overhead signs and flashing beacons
3) Pedestrian-activated flashing LED signs
4) Rectangular rapid flashing beacons (RRFBs). The device is subject to interim approval. INDOT is responsible for maintaining an inventory of all RRFB locations in Indiana; therefore, the Traffic Administration Office should be notified of all proposed RRFB installations at Final Plans/Stage 3 submittal.

**Level 5**  **Traffic Signals and Grade Separation**
1) Pedestrian hybrid beacon (“HAWK Signal”)
2) Pedestrian-actuated traffic signal
3) Grade-separated crossing

**RECOMMENDED TREATMENT OF SHARED USE PATH AND ROADWAY INTERSECTION**

Figure 51-7 O (Pg. 2 of 2)
Intersection traffic control devices as warranted depending on conditions. See MUTCD

MIDBLOCK TYPE CROSSING

Figure 51-7P
TYPICAL REALIGNMENT OF DIAGONAL SHARED-USE-PATH CROSSING AT ROADWAY INTERSECTION

Figure 51-7Q
ADJACENT SHARED-USE-PATH TO ROADWAY INTERSECTION WITH ANOTHER ROADWAY

Figure 51-7R
Direction of narrow-wheeled-vehicle travel
Widen shared-use path to permit right angle crossing
Maintain Width
Shoulder of shared-use path
NARROW-WHEELED-VEHICLE SAFE RAILROAD CROSSING

Figure 51-7S
**REFUGE ISLAND AT ROADWAY INTERSECTION**

**Figure 51-7T**

- **W (offset)** = \( \frac{Y}{2} \)
- **L** = \( \frac{W V^2}{60} \), Where \( V \leq 40 \) mph
- **L** = \( W V \), Where \( V \geq 45 \) mph

**X** = Length of island. Should be 6 ft or greater.

**Y** = Width of refuge:
- 6 ft = poor
- 8 ft = satisfactory
- 10 ft = good
STRIPING AT MOTOR-VEHICLE BARRIER POST

Figure 51-7U
<table>
<thead>
<tr>
<th>Pavement Classification</th>
<th>R1</th>
<th>R2 or R3</th>
<th>R4</th>
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<tr>
<td>$E_h$ (ft-cd)</td>
<td>1.4</td>
<td>2.0</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Notes:

- **R1** = portland-cement concrete
- **R2** = asphalt, aggregate consists of minimum 60% gravel passing 3/8-in. sieve
- **R3** = asphalt, rough texture (typical highway)
- **R4** = asphalt, smooth texture

The maximum uniformity ratio is 3:1.


MINIMUM AVERAGE MAINTAINED ILLUMINATION, $E_h$
BASED ON PAVEMENT CLASSIFICATION

Figure 51-7V
(a) MINIMUM LENGTH REQUIRED

(b) MINIMUM OVERLAP REQUIRED

SOUND BARRIER PLACEMENT

Figure 51-9A
AVOID LARGE COLUMN PROTRUSIONS ON WALL ADJACENT TO TRAFFIC LANE

ACCEPTABLE

AVOID FACING WHICH MAY BECOME SAFETY HAZARDS WHEN HIT

FACING SET INTO RECESS

SOUND BARRIER PROTRUSIONS

Figure 51-9B
<table>
<thead>
<tr>
<th>Highway Type and Traffic Conditions</th>
<th>Width, $W$, of All-Weather Surface of Turnout or Available Shoulder at Mailbox (ft)</th>
<th>Distance Roadside Face of Mailbox is to be Offset Behind Edge of Turnout or Usable Shoulder (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Preferred</td>
<td>Minimum</td>
</tr>
<tr>
<td>Rural highway AADT &gt;10,000</td>
<td>&gt; 12</td>
<td>12</td>
</tr>
<tr>
<td>Rural highway 1,500 &lt; AADT ≤ 10,000</td>
<td>12</td>
<td>10</td>
</tr>
<tr>
<td>Rural highway 400 &lt; AADT ≤ 1500</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>Rural road AADT ≤ 400</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>Residential street without curb or all-weather shoulder</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>Curbed residential street</td>
<td>Not Applicable</td>
<td></td>
</tr>
</tbody>
</table>

AADT = Average Annual Daily Traffic  
vpd = vehicles per day

* If a turnout is provided, this may be reduced to zero.

**SUGGESTED GUIDELINES FOR LATERAL PLACEMENT OF MAILBOXES**

*Figure 51-11A*
ROUNDABOUT ELEMENTS

Figure 51-12A
SINGLE-LANE ROUNDABOUT

Figure 51-12D
MULTILANE ROUNDABOUT,
ONE APPROACH ROADWAY 4 LANES

FIGURE 51-12E
MULTILANE ROUNDABOUT, BOTH APPROACH ROADWAYS 4 LANES

Figure 51-12F
## DAILY SERVICE VOLUME
FOR 4-LEG ROUNDABOUT

<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>Design-Year AADT</th>
</tr>
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<tr>
<td>Single</td>
<td>&lt; 25,000</td>
</tr>
<tr>
<td>Two</td>
<td>40,000 &lt; AADT &lt; 45,000</td>
</tr>
<tr>
<td>Three</td>
<td>&lt; 60,000</td>
</tr>
</tbody>
</table>

Figure 51-12H
<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>Combined Vehicles per Hour For All Approaches</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single</td>
<td>≤ 2,000</td>
</tr>
<tr>
<td>Two</td>
<td>≤ 4,000</td>
</tr>
<tr>
<td>Three</td>
<td>≤ 7,000</td>
</tr>
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</table>

HOURLY SERVICE VOLUME
FOR 4-LEG ROUNDABOUT

Figure 51-12 I
<table>
<thead>
<tr>
<th>Geometric Parameter</th>
<th>Single-Lane Entry</th>
<th>Two-Lane Entry</th>
<th>Three-Lane Entry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Entry Width, $E$ (ft)</td>
<td>18 to 22</td>
<td>24 to 28</td>
<td>34 to 40</td>
</tr>
<tr>
<td>Effective Flare Length, $L'$ (ft)</td>
<td>15 to 300, if required for capacity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Half Width, $V$ (ft)</td>
<td>Travel-lane width approaching roundabout prior to flared section, or width between lane markings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Entry Radius, $R$ (ft)</td>
<td>55 to 90</td>
<td>55 to 100</td>
<td>65 to 100</td>
</tr>
<tr>
<td>Entry Angle, $\phi$ (deg)</td>
<td>16 to 30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Circulatory-Roadway Width</td>
<td>1.0 to 1.2 times the width of the widest roundabout entry</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Exit Radius (ft)</td>
<td>200 to 1000. Exit-curve radius should be longer than entry-curve radius. R3 speed should be higher than R2 speed.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESIGN-VALUES RANGES**

*Figure 51-12J*
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Mini-Roundabout</th>
<th>Urban Area</th>
<th>Rural Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Compact</td>
<td>Single-Lane</td>
</tr>
<tr>
<td>ICD, ft</td>
<td>45 to 110</td>
<td>80 to 110</td>
<td>100 to 140</td>
</tr>
<tr>
<td>Splitter-Island Treatment</td>
<td>Raised if poss., crosswalk cut if raised</td>
<td>Raised, with crosswalk cut</td>
<td>Raised and extended, with crosswalk cut</td>
</tr>
</tbody>
</table>

**BASIC DESIGN CHARACTERISTICS**

*Figure 51-12K*
KEY DESIGN PARAMETERS

Figure 51-12L

V = Half width
E = Entry width
L' = Effective flare length
R = Entry Radius
D = Inscribed Circle Diameter
Ø = Entry Angle
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Half Width, $V$ (ft)</td>
<td>Width of the roadway used by approaching traffic upstream of changes in width associated with the roundabout. It is not more than half of the total roadway width. If the facility has a marked bicycle lane, this width is to the white line. If there is no marked bicycle lane, this width is from the right-side curb face to the splitter-island curb face or marked centerline on the left side.</td>
</tr>
<tr>
<td>Entry Width, $E$ (ft)</td>
<td>Width where it meets the inscribed circle. It is measured perpendicularly from the outside-curb face to the inside-curb face at the splitter island’s nearest point to the yield line.</td>
</tr>
<tr>
<td>Effective Flare Length, $L'$ (ft)</td>
<td>Half the total distance between $V$ and $E$. At $L'$ the approach-roadway width equals the average of $V$ and $E$. The flare should be developed uniformly, without a sharp break where it starts. Full flare length equals $2L'$.</td>
</tr>
<tr>
<td>Entry Radius, $R$ (ft)</td>
<td>The outside curbs’ minimum radius of curvature at the entry.</td>
</tr>
<tr>
<td>Entry Angle, $\varphi$, (deg)</td>
<td>Used in the empirical formula.</td>
</tr>
<tr>
<td>Inscribed-Circle Diameter, ICD (ft)</td>
<td>The basic parameter used to define a roundabout’s size. It is measured between the outer edges of the circulatory roadway.</td>
</tr>
</tbody>
</table>

**KEY ROUNDABOUT-DESIGN PARAMETERS**

*Figure 51-12M*
GEOMETRIC DESIGN PARAMETERS

Figure 51-12N
CAPACITY VS. INSCRIBED-CIRCLE DIAMETER

Figure 51-12 O
Pedestrian- or bicycle-use facility, not a bypass lane.

Full-bypass lane

RIGHT-TURN BYPASS LANE

Figure 51-12P
LANE-CONFIGURATION SKETCH EXAMPLE

Figure 51-12S
• (a) is utilized with a high-volume right-turn movement.

• (b) is utilized with a high-volume left-turn movement.

LANE MARKINGS

Figure 51-12T
<table>
<thead>
<tr>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle speed is reduced, compared to that for another intersection type.</td>
<td>Vehicle traffic is yield controlled, so it does not necessarily come to a full stop. Therefore, a pedestrian can be hesitant at first to use the crosswalk.</td>
</tr>
<tr>
<td>A pedestrian has fewer conflict points that at another intersection type.</td>
<td>A roundabout can be unsettling to a pedestrian, depending on age, mobility, visual impairment, or ability to judge gaps in traffic.</td>
</tr>
<tr>
<td>A pedestrian is responsible for judging crossing opportunity. This is still regarded as an advantage, though it requires more alertness.</td>
<td>A pedestrian, at first glance, can have to adjust to roundabout operation. This includes the crosswalk location, which is behind the first stopped vehicle, or 6 m from the yield point.</td>
</tr>
<tr>
<td>The splitter-island refuge allows a pedestrian to cross entering and exiting traffic flows separately, and thus simplifies the task of crossing the roadway.</td>
<td></td>
</tr>
<tr>
<td>Crossing can be accomplished with less waiting time than at a signalized intersection.</td>
<td></td>
</tr>
</tbody>
</table>

**ROUNDABOUT ADVANTAGES AND DISADVANTAGES FOR PEDESTRIANS**

*Figure 51-12U*
ROUNDABOUT FEATURES

Figure 51-12V
BICYCLE-RAMP ENTRANCE AND EXIT

Figure 51-12W
EVALUATION AND DESIGN PROCESS

Figure 51-12X
<table>
<thead>
<tr>
<th>ELEMENT</th>
<th>SAFETY</th>
<th>CAPACITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wider entry</td>
<td>Reduced</td>
<td>Increased</td>
</tr>
<tr>
<td>Longer entry radius</td>
<td>Reduced</td>
<td>Increased</td>
</tr>
<tr>
<td>Smaller entry angle</td>
<td>Reduced</td>
<td>Increased</td>
</tr>
<tr>
<td>Larger angle between entries</td>
<td>Reduced</td>
<td>Increased</td>
</tr>
<tr>
<td>Longer flare length</td>
<td>Increased</td>
<td>Reduced</td>
</tr>
<tr>
<td>Wider circulatory roadway</td>
<td>Reduced</td>
<td>Increased</td>
</tr>
<tr>
<td>Larger inscribed-circle diameter</td>
<td>No Change</td>
<td>Increased</td>
</tr>
</tbody>
</table>

**EFFECTS OF DESIGN ELEMENTS ON SAFETY AND CAPACITY**

Figure 51-12Y
RADIUS | DESCRIPTION
--- | ---
Entry-path, R1 | Minimum radius on the fastest through path prior to the yield line. This is not the same as the entry radius.
Circulating-path, R2 | Minimum radius on the fastest through path around the central island.
Exit-path, R3 | Minimum radius on the fastest through path into the exit.
Left-turn path, R4 | Minimum radius on the path of a conflicting left-turn movement.
Right-turn path, R5 | Minimum radius on the fastest path for a right-turning vehicle.

ROUNDABOUT-RADIUS DESCRIPTIONS

Figure 51-12AA
A. 5 ft from left side face of curb, or 3 ft from painted center line or flange line of curb and gutter on each approach and exit.

B. 5 ft from face of curb on driver’s right side at each entry and exit.

C. 5 ft from central island face of curb.

D. Not less than 165 ft from the inscribed-circle diameter (ICD). This distance typically is 165 ft but can be greater depending on how a driver approaches the yield line at high speed.

OFFSETS FOR FASTEST PATH

Figure 51-12CC
SPLINE CURVE THROUGH MOVEMENT

Figure 51-12DD
SPLINE CURVE BETWEEN CURB OFFSET AND CURB

Figure 51-12FF
R5 SPLINE EXAMPLE

Figure 51-12 I I
Optional Separation Between Lanes (typ.)

40° to 60°
20° to 30°

Chevron markings (typ.)

ENTRY DEFLECTION

Figure 51-12JJ
TYPICAL SPLITTER ISLAND

Figure 51-12KK

- A Concrete center curbs type C and D (typ.)
- B Sidewalk curb ramps type L
a) The radius should be measured over a distance of 65 to 80 ft. It is the minimum that occurs along the approach entry path near the yield point but not more than 165 ft in advance of it.

b) Beginning point is 3 ft from pavement marking with no curb face present and is 5 feet from the left curb face, if raised median curb at a point not less than 165 ft from the yield point. This point is a continuation of a vehicle path, not a point with deflection.

c) Vehicle entry path curvature.

DETERMINATION OF ENTRY-PATH CURVATURE

Figure 51-12LL
**PATH-OVERLAP CHECK METHOD**

Figure 51-12NN
PATH-OVERLAP AVOIDANCE TECHNIQUES

Figure 51-12 OO

LEGEND:
LW = Lane width
CW = Circulatory width

Median widened toward exit lanes to maximize entry deflection
Typical taper rate
Initial small radius entry curve
Second larger radius entry curve or tangent
Outside edge of circulatory roadway

Central Island
ICW
CW/2
MULTI-LANE ENTRY DESIGN

Figure 51-12PP
Radius to face of curb should be that to maintain lateral clearance for signing per MUTCD.

Curb face offset 6'-0" desirable 4'-0" minimum

Match shoulder width

Median-island pavement marking

Concrete curb and gutter

Minimum shifting taper based on horizontal shift and posted speed limit.

Min. Right side curb length to be 25' prior to bike ramp or 100' prior to yield line, whichever is greater.

Curbed length equal to AASHTO Policy on Geometric Design of Highways and Streets, Exhibit 10-73 deceleration length, L.

Design speed ≥ 50 mph

Design speed ≤ 45 mph

HIGH-SPEED ROUNDAABOUT APPROACH

Figure 12QQ
REGULATORY SIGNS

Figure 51-12RR
WARNING SIGNS

Figure 51-12SS
SAMPLE DESTINATION SIGNS

Figure 51-12TT
SAMPLE EXIT SIGNS

Figure 51-12 V V
<table>
<thead>
<tr>
<th>Posted or 85\textsuperscript{th}-Percentile Speed (mph)</th>
<th>Minimum-Visibility Distance (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>155</td>
</tr>
<tr>
<td>30</td>
<td>200</td>
</tr>
<tr>
<td>35</td>
<td>250</td>
</tr>
<tr>
<td>40</td>
<td>305</td>
</tr>
<tr>
<td>45</td>
<td>360</td>
</tr>
<tr>
<td>50</td>
<td>425</td>
</tr>
<tr>
<td>55</td>
<td>495</td>
</tr>
</tbody>
</table>

**MINIMUM VISIBILITY DISTANCE**

Figure 51-12WW
ILLUMINATED BOLLARDS

Figure 51-12XX
In considering the modern roundabout as an intersection alternative, a number of common misconceptions are too-often presumed by the public, elected officials, consultants, or transportation experts who are unfamiliar with this type of intersection control.

Some facts concerning the modern roundabout are as follows:

1. The modern roundabout is significantly different than an old-style traffic circle or rotary.
2. If designed properly, a modern roundabout is safer than a traffic circle or a traditional signalized intersection, and is often used as a calming effect to slow traffic.
3. A roundabout increases a road’s capacity as it can handle high traffic volume. It can also require fewer lanes or reduced median width as it can reduce congestion or backups.
4. A roundabout is an effective treatment for a rural intersection where signalization may not be appropriate.
5. A roundabout can provide adequate downstream gaps for motorists entering the roadway from side streets or drives.
6. A roundabout is inexpensive to operate, is easily modifiable, and is low-maintenance.
7. A roundabout safely accommodates high volumes of pedestrians and bicyclists.
8. A roundabout reduces speed and causes fewer backups than stop-and-go traffic.
9. A roundabout can be landscaped to be aesthetically attractive.

THE FACTS ABOUT ROUNDABOUTS

Figure 51-12YY
CHAPTER 308

Special Design Elements

<table>
<thead>
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<th>Design Memorandum</th>
<th>Revision Date</th>
<th>Sections Affected</th>
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<tbody>
<tr>
<td>14-07</td>
<td>Apr. 2014</td>
<td>Ch. 47</td>
</tr>
<tr>
<td>16-18</td>
<td>Apr. 2016</td>
<td>Section 47-1.05, Figure 47-1A</td>
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CHAPTER 47

Railroad-Highway Grade Crossings

<table>
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<th>Revision Date</th>
<th>Sections Affected</th>
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<td>14-07</td>
<td>Apr. 2014</td>
<td>Entire Chapter 47</td>
</tr>
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<td>16-18</td>
<td>Apr. 2016</td>
<td>Section 47-1.05, Figure 47-1A</td>
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  47-1.04  Sight Distance ........................................................................................................... 4
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<tr>
<td>47-1A</td>
<td>Near Terminus [Added Apr. 2016]</td>
</tr>
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CHAPTER 47

RAILROAD-HIGHWAY GRADE CROSSINGS

[Rev. Apr. 2014]

47-1.0 DESIGN CRITERIA

The geometric design of railroad-highway grade crossings should be made jointly when determining the warning devices to be used. When only passive warning devices such as signs and pavement markings are used, the highway drivers are warned of the crossing location but must determine for themselves whether or not there are train movements for which they should stop. On the other hand, when active warning devices such as flashing light signals or automatic gates are used, the driver is given a positive indication of the presence or the approach of a train at the crossing.

Traffic control devices for railroad-highway grade crossings consist primarily of signs, pavement markings, flashing light signals, and automatic gates. Criteria for design, placement, installation, and operation of these devices are covered in the IMUTCD and the FHWA Railroad-Highway Grade Crossing Handbook.

A railroad-highway grade crossing on a 3R or 4R project should be in accordance with the design criteria as described in the following sections.

47-1.01 Horizontal Alignment

The highway should intersect the tracks at a right angle with no nearby intersections or driveways. To the extent practical, crossings should not be located on either highway or railroad curves.

The AASHTO Policy on Geometric Design of Highways and Streets (Greenbook) specifies that where highways that are parallel with main tracks intersect highways that cross the main tracks, there should be sufficient distance between the tracks and the highway intersections to enable highway traffic in all directions to move expeditiously. Where physically restricted areas make it impractical to obtain adequate storage distance between the main track and a highway intersection, the following should be considered:

1. interconnection of the highway traffic signals with the grade crossing signals to enable vehicles to clear the grade crossing when a train approaches; and
placement of a “Do Not Stop on Track” sign on the roadway approach to the grade crossing.

**47-1.02 Vertical Alignment**

The approach elevation should be the same elevation as the top of rails for a distance of 2 ft outside the rails.

The surface of the highway should not be more than 3 in. higher or lower than the top of the nearest rail at a point 30 ft from the rail unless track superelevation dictates otherwise.

**47-1.03 Cross Section**

There should not be a raised curb or obstruction within 10 ft of the rail. This guidance is to ensure the railroad company has access to the area adjacent the rail.

**47-1.04 Sight Distance**

Sight distance is a primary consideration at crossings without train-activated warning devices. Adequate stopping sight distance is needed so that a driver can see an approaching train and have sufficient distance to stop safely. Recommended sight distance values and additional discussion of sight distance at railroad-highway grade crossings can be found in the 2011 *Greenbook*, Section 9.12.

**47-1.05 Crossing Warning Devices [Rev. Apr. 2016]**

The Department must comply with 23 CFR §646.214 regarding railroad-highway grade crossing improvement for each Federal-aid project. The Department’s philosophy is to appropriately allocate limited resources and maximize system-wide improvements. This approach targets investment decisions to the roadway system as a whole. To reduce crash risk, uniform warning device configurations at all railroad-highway grade crossings are the best practice.

Where a railroad-highway grade crossing is located within or near the terminus of the project limits, the crossing must be evaluated for inclusion of railroad warning devices in the project scope of work. The limits also apply to maintenance of traffic. Near the terminus or “near terminus” is defined below. The Department’s *Policy for Railroad-Highway Grade Crossing Warning Devices* provides the evaluation procedures for determining the level of warning.
protection, either passive or active (train-activated) that is required. The district or Central Office railroad coordinator should be contacted to assist in the evaluation. The evaluation recommendation must be reviewed and approved by the Utilities and Railroads Division Senior Rail Projects Engineer.

Warning Devices

A basic passive device upgrade is required for all projects that include a railroad-highway grade crossing within or near the terminus of a project. A passive device upgrade consists of replacing the existing cross bucks with high retro-reflectivity cross bucks, adding reflectorized striping to the post, and installation of a yield or stop sign, installing any required pavement markings, and installing or upgrading advance warning signage. Note that per the IMUTCD an engineering study is required prior to the installation of a stop sign.

If active protection is deemed necessary based on the policy, then the upgrade or installation of gates, flashing lights, overhead cantilever, warning bell, and constant warning time (CWT) circuitry is the minimum acceptable level of active warning. No incremental or intermediate improvements to active warning devices are allowed.

Near Terminus

The decision point used to determine if the location of the crossing is near the terminus of a project is based on the transverse pavement markings from the nearest rail. The markings are shown in the INDOT Standard Drawing 808-MKPM-06. The decision point, or near terminus, is the leading perpendicular line of the railroad crossing pavement marking. The distance from the nearest rail to the near terminus varies with design speed, and is shown as dimension D in Figure 47-1A. Where the project limits are within the distance D, the crossing must be included in the project scope of work. The near terminus applies regardless of the actual presence of pavement markings on the roadway.

47-2.0 COORDINATION WITH RAILROAD COMPANY

The designer should contact the Department railroad coordinator when there is a railroad within or near the proposed highway project. The railroad coordinator will determine the need for the railroad company’s attendance at the preliminary field check. See Chapter 105 for railroad coordination.
### Controlling Dimension For Determining Crossing Inclusion

<table>
<thead>
<tr>
<th>Roadway Design Speed</th>
<th>D = Distance from nearest rail to controlling pavement marking*</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 35 mph</td>
<td>131 Feet</td>
</tr>
<tr>
<td>40 mph</td>
<td>156 Feet</td>
</tr>
<tr>
<td>45 mph</td>
<td>206 Feet</td>
</tr>
<tr>
<td>50 mph</td>
<td>281 Feet</td>
</tr>
<tr>
<td>55 mph</td>
<td>356 Feet</td>
</tr>
<tr>
<td>60 mph</td>
<td>431 Feet</td>
</tr>
</tbody>
</table>

* Where the project limits are within the distance D from the nearest rail, the crossing must be included in the project scope.

Example: The design speed is 45 mph and the project limits are 200 ft from the nearest rail. The crossing must be included in the project scope because 200 ft is less than D (206 ft).

**NEAR TERMINUS DEFINITION**

**Figure 47-1A**